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ADVANCED COMPACTION QUALITY CONTROL

by

Carlos Zambrano, M.S. Student
Vincent Drnevich, P.E., Ph.D., Professor
Philippe Bourdeau, Ph.D., Assoc. Professor

School of Civil Engineering
Purdue University

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16. Abstract

This Synthesis Study gathered the current knowledge on the topic of Intelligent Compaction and provides INDOT with recommendations for action. It appears that totally embracing this technology at this time is premature. On the other hand, it also appears that prudent use of this technology could provide tremendous benefit-to-cost ratios over the long term and position Indiana as a leader among the States in adopting it. After examining the technological developments over the last decade it is concluded that Intelligent Compaction is a near certainty in the future. The big question is: When is the right time to begin working on it? The answer is now! There is much activity in this area on the part of the FHWA and NCHRP. However, some major questions exist on which technology to adopt, how to accurately calibrate it, and what do the test results mean for design of pavements and other highway structures.

Adoption of Intelligent Compaction will make obsolete current quality control processes that are principally based on spot tests that lead to delays in construction work, due to laborious testing and time lags from testing to having to be done in the laboratory or field office. Furthermore, these tests normally have a sample volume of about 1000 cm^3, being an unreliable way to represent the compaction results of the entire worked area.

A far more important consequence will be the increased uniformity of the compacted material. It can be argued that premature failures in pavements and embankments occur primarily from “pockets” of material with anomalous properties that create “weak links” or discontinuities which give rise to distress. Intelligent compaction could eliminate the existence of these. Continued use of current spot methods, even with heavily increased spatial frequency, will be neither foolproof nor economical.

This report provides both short-term and long-term recommendations. The short-term recommendations are to modify existing INDOT specifications for proof-rolling to make them easier to interpret and consistent with other states such as Ohio, Minnesota, Wisconsin, New York, Arizona, Iowa, North Carolina and Colorado.

The long-term recommendations are to INDOT are to participate in the pool funded project of FHWA. It also is recommended that the highway contractors of Indiana become involved to show them the benefits associated with the technology that will provide exceptional value to all who understand and make use of it.

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CHAPTER 1 - INTRODUCTION

1.1 Background

With the prospect of constructing I-69 from Indianapolis to Evansville, the State of Indiana has an opportunity to implement 21st Century quality control procedures for earthworks that will provide superior performance and greatly extend the service of pavements and other structures supported by and built of earthen materials. Much of the technology and infrastructure are already in existence for implementing quality control for nearly 100 percent of earthworks. The technologies include: Global Positions Systems (GPS), machine sensors, reliable transduction systems, microprocessors, robust and inexpensive mass storage, and convenient interfaces for accessing and using acquired information. When applied to earthworks, these technologies are generally referred to as Intelligent Compaction. They frequently utilized systems that are attached to key compaction equipment that monitor the equipment performance and use the results to indicate to the operator whether or not the soil beneath the compactor has been compacted sufficiently to meet specifications. With this technology, compaction specifications eventually will replace the conventional water content and density specifications now in widespread use with those required for the actual performance of pavements and embankments such as resilient modulus, stiffness, and shear strength.

This synthesis report includes a review of the literature and results of discussions with geotechnical engineering researchers and major equipment manufacturers such as Caterpillar, Dynapac, Ammann, Geodynamik and Bomag. The manufacturers are working diligently to perfect these technologies and are introducing them as a means of promoting the sale of their equipment. According to Nordfelt (2005), "about 80% of the rollers sold in Europe have some type of continuous compaction control."

The Synthesis Study gathered the current knowledge on the topic of Intelligent Compaction and provides INDOT with recommendations for action. It appears that it may be premature at this time to fully engaging this technology. On the other hand, prudent use of this technology could provide tremendous benefit-to-cost ratios over the long term and position Indiana as a leader among the States in adopting it. Examination of the technological developments over the last decade, leads to the conclusion that Intelligent Compaction is a near certainty in the future. The big question is: When is the right time to begin working on it?

Adoption of Intelligent Compaction will make obsolete current quality control processes that are principally based on spot tests that lead to delays in construction work, due to laborious testing and time lags from testing to having to be done in the laboratory or field office. Furthermore, these tests normally have a sample volume of about 1000 cm^3, being an unreliable way to represent the compaction results of the entire worked area.

A far more important consequence will be the increased uniformity of the compacted material. It can be argued that premature failures in pavements and embankments occur primarily from "pockets" of material with anomalous properties that create "weak links" or discontinuities which give rise to distress. Intelligent compaction could eliminate the existence of these. Continued use of current spot methods, even with heavily increased spatial frequency, will be neither foolproof nor economical.

Up to the moment several countries, particularly Germany, Austria and Sweden, have included advanced quality control procedures in the engineering practice that have led them to minimize the amount and frequency of concrete or asphalt-paving rework in an efficient way (Sandstrom et al, 2004). These new processes use feedback techniques in order to achieve the highest possible efficiency from the first to the last pass of the roller over the entire area, making possible the assessment of 100% of the work area.

Research on these techniques is escalating, especially that on the methods associated with the automatically adaptation of compaction equipment to the varying soil conditions during compaction process and on the methods that estimate stiffness parameters of the complete compacted area through the soil-roller dynamic interaction. A fairly in-depth discussion of them is included in this report. The application of these methods will surely lead to reduce the maintenance and repair costs of roads, and increase the efficiency in the compaction work in the State of Indiana.

1.2 General scope of the work

This report provides the results of a synthesis study that included a literature review of the State of the Art in compaction quality control procedures and the current advanced methods of Intelligent Compaction in the US and other countries. The report summarizes the current state of the art as well as emerging technologies for compaction control (Intelligent Compaction). The report makes recommendations to INDOT for both short-term and long-term actions for improving compaction control.

The major tasks of the research project were:

- Search and study of the existing advanced quality control procedures in use and under development in the world. The search was done through the National Transportation Library (TRIS Online), journals, and international conferences. Additionally, contacts were made with earthwork equipment industry people and engineers that are involved in the development and application of new compaction technology from the US and other countries.

- Evaluated compaction quality control and intelligent compaction procedures in US and other countries, identifying the advantages and limitations of each.

- Based on the above study and analyses, the report includes a course of action for INDOT to pursue for critical evaluation, calibration, pilot implementation, and eventual adoption for practical use.

CHAPTER 2 - PAVEMENT DESIGN

2.1 Introduction

The principal design procedure for highways in the US is the AASHTO Guide for the Design of Pavement Structures. The design guides from the early 1960's through 1986 were based on empirical performance equations developed at the AASHO Road Test conducted near Ottawa, Illinois, in the late 1950's. Since the time of the AASHO Road Test, there have been many significant changes in trucks, axles, loads, truck traffic, materials, construction, rehabilitation, and performance criteria, which require a design procedure that accounts for particular load conditions, materials, and direct consideration of climatic effects on pavement performance (ARA, Inc. Consultant Division, 2004)

The AASHTO Joint Task Force on Pavements, in cooperation with the National Cooperative Highway Research Program (NCHRP) and Federal Highway Administration (FHWA), began the development of the guide for mechanistic-empirical design of new and rehabilitated pavement structures in 1996. A research guide version is available for evaluation at www.trb.org/mepdg.

The evolution from the method based only on experience to the methods based on a combination of mechanics theory and field observations led to the use of the resilient modulus (M_R), which is one of the fundamental parameters of the mechanistic-empirical approach.

This chapter first presents a brief description of the stress state, and stress and strain range in pavement structures under working conditions. This information is fundamental to the estimation of the Resilient Modulus, $M_{R,}$ because this parameter is stress dependent. Finally, a review is given of the definition of M_R in pavement design, and the main factors that control M_R in clayey and sandy soils. This Chapter will be the basis of the discussion on M_R estimation through laboratory tests (Section 2.4) and field tests, including the intelligent compaction technology (Chapter 3 and 4).

2.2 Cyclic load on Pavements

The subgrade and pavement structure are subjected to cyclic loads induced by moving wheels that induce a complex stress pattern.

As shown in Figures 2.1 and 2.2, an element of material within the pavement structure or the subgrade experiences normal and shear stresses that change with time as the load passes by. During the loading cycle variations in stress magnitude are accompanied by rotation of their principal axes. For example, when a vehicle wheel is far away from a fixed point, the stress at that point is geostatic. As the wheel approaches the vertical stress increases up to a maximum value when the wheel is just above the point, at this point the shear stresses are zero. When the vehicle advances, the wheel moves away from the point and the normal stress decreases and the shear stress increases. As the wheel moves away, both, the shear and normal stress will dissipate. (See Figures. 2.1 and 2.2.)

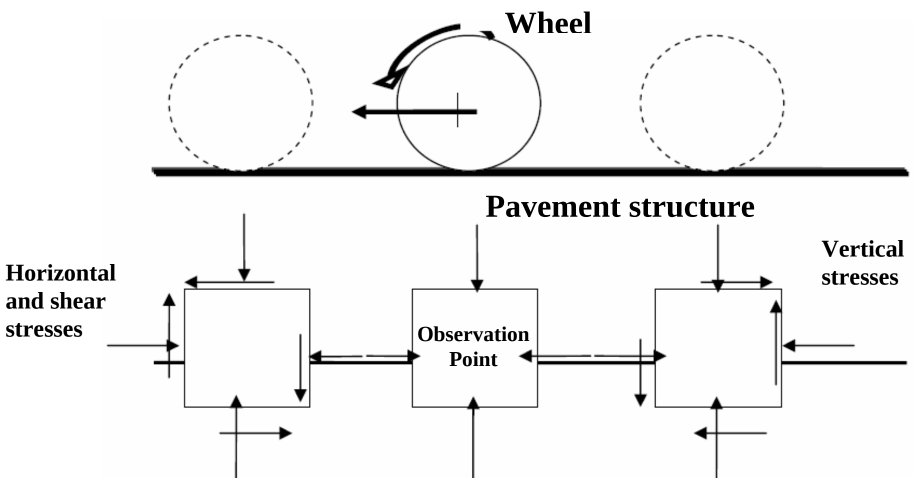

Figure 2.1 – Stress on subgrade and pavement structure on a given element (Brown, S.F. 1996, and Anguas, et al, 2001).

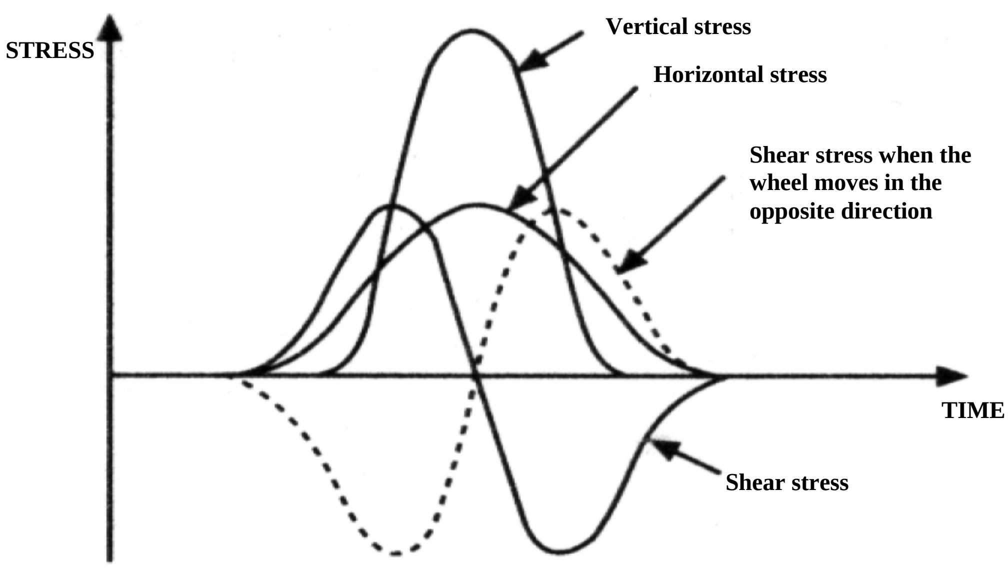

Figure 2.2 – Stress variation with time at a given point (Brown, S.F, 1996).

However, the critical loading condition is not always represented by the maximum normal load that occurs at the points beneath the wheel. At some radial distance from the centerline of loading, the horizontal component of the dynamic wheel load can become greater in magnitude than the vertical component. In that case the extension type of loading can be more critical at the top of the pavement base (Seyhan et al., (2000)).

Based on finite element modeling, Seyhan et al., (2000) observed that extension states primarily occurred in the granular layer when the initial compressive stress was less than 3 lb/in^2 (21 kPa). Such magnitudes of initial compressive stress were reported previously to exist in the compacted granular layers, which offset any low magnitude of horizontal tensile stresses and provide adequate radial confinement away from the wheel load (Seyhan et al, 2000).

The main implication of the stress states described above is that M_R is a stress dependent and anisotropic property of the material. Section 2.4 briefly discusses the characterization of materials through laboratory testing.

2.3 Stress and strain levels in Pavements

In pavement engineering, the soil is subjected to a stress-controlled environment. This means that the strain level in the soil depends on the applied stress and the deformability properties of the materials.

The pressure applied to pavement surface is of the order of 30 lb/in^2 (200 kPa) for automobile tires, 70 lb/in^2 (500 kPa) for truck tires, and 250 lb/in^2 (1700 kPa) for airplanes tires. The stress at the subgrade level is function of the thickness of the pavement structure. If the pavement is thin, the cyclic deviator stresses at the subgrade level are high, and the opposite applies for thick pavement structures. The dissipation of stress in the pavement is due mainly to stiffness contrast between the pavement and subgrade (See Figure 2.3). The stress in the subgrade is much less than it would be without a stiff pavement layer.

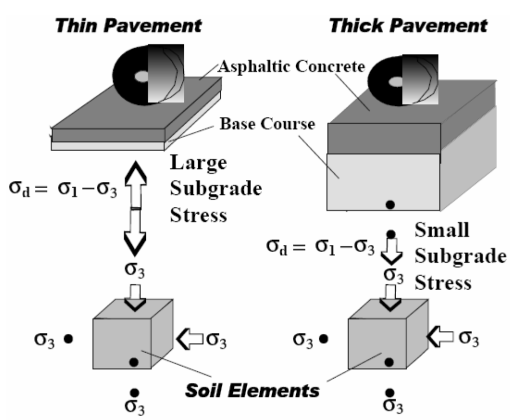

Figure 2.3 – Relative subgrade stress levels for different pavement thickness (Hopkins, T.C, et al., (2001)).

Under normal service conditions, soils and granular materials beneath pavements are subjected to a large number of loads at stress levels well below their shear strength. This condition is associated to long term surface deformation measured in millimeters if not tenths of millimeters. Typical strain levels at the top of the subgrade are around 0.1% (Briaud et al, 2003).

In order to gain insight on magnitudes of stresses and strains in pavement structures due to working loads, several experimental research programs have included full scale tests. Figure 2.4 presents stress measurements with time for a point on a pavement associated with the transit of a truck on gravel road and on asphalt pavement. It is evident that the function of the asphalt layer reduces the vertical stresses.

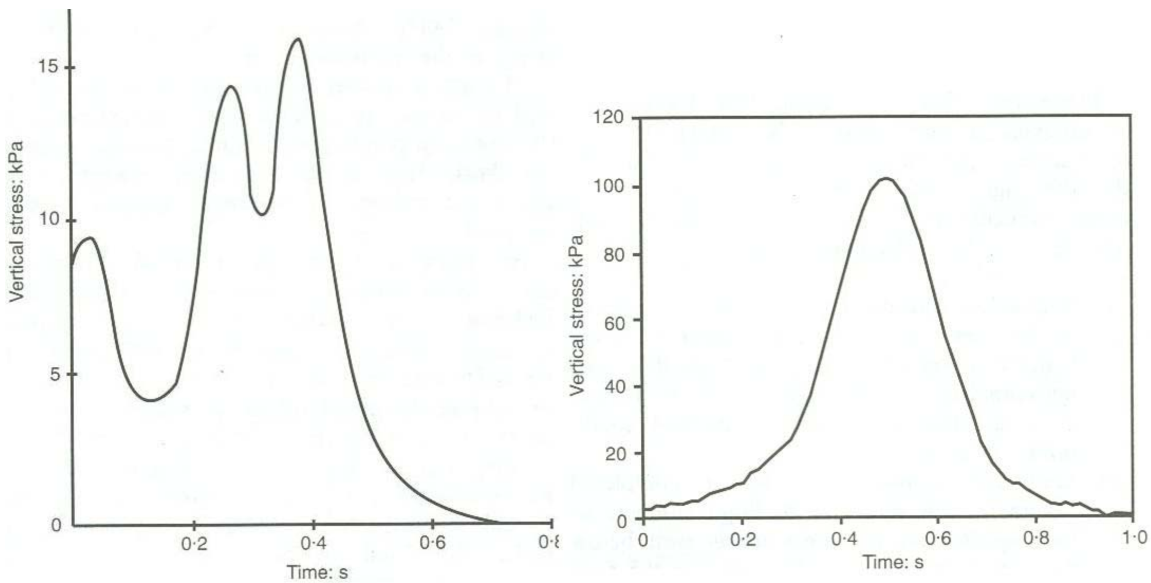

Figure 2.4 – In situ vertical stress measurements in subgrades: a) below 6.5 in (165 mm) asphalt construction; b) below 14 in (350 mm) granular layer (Brown, S.F, 1996) [Note that the vertical scales on these two plots are significantly different.]

Based on full scale test results, researchers have proposed that the stress pulses induced by a wheel in movement along pavements is quite well represented by semi-sinusoidal (sometimes referred to as haversine) or triangular functions, with duration given by the vehicle velocity and the vertical distance below the pavement surface.

Figure 2.5 indicates that the pulse duration increases as the vehicle velocity decreases and as the depth to the point of observation decreases. For practical purpose Huang (1993) suggests the use of a wave form function with duration of 0.1 second with a rest period of 0.9 second. The AASHTO T292 standard method for resilient modulus recommends triangular and rectangular wave forms with load duration, associated with pavement design speed and depth below pavement surface, ranging from 0.05 to 0.15 seconds with fixed cycle durations between 1 and 3 seconds.

The duration of load application has a small effect in granular materials, and some effect on fine-grained soils, depending on the water content, while in asphaltic materials the effect is considerable.

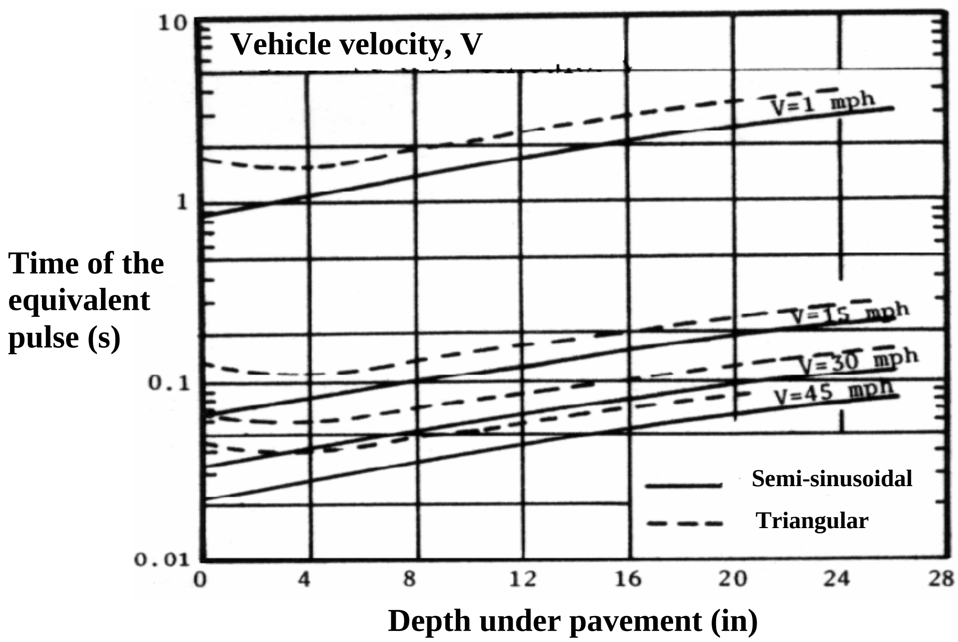

Figure 2.5 – Application load time associated to a semi-sinusoidal and triangular pulse stress (Huang, (1993)).

2.4 Resilient Modulus

The resilient modulus is used to determine the elastic part of the material response under repeated loading-unloading conditions caused by traffic (See Figure Fig 2.6). It is based on recoverable strain under repeated loads.

$$M_R = \sigma_D / \varepsilon_R \tag{2.1}$$

Where M_R is the resilient modulus, σ_D is the deviator stress, and ε_R is the recoverable axial strain.

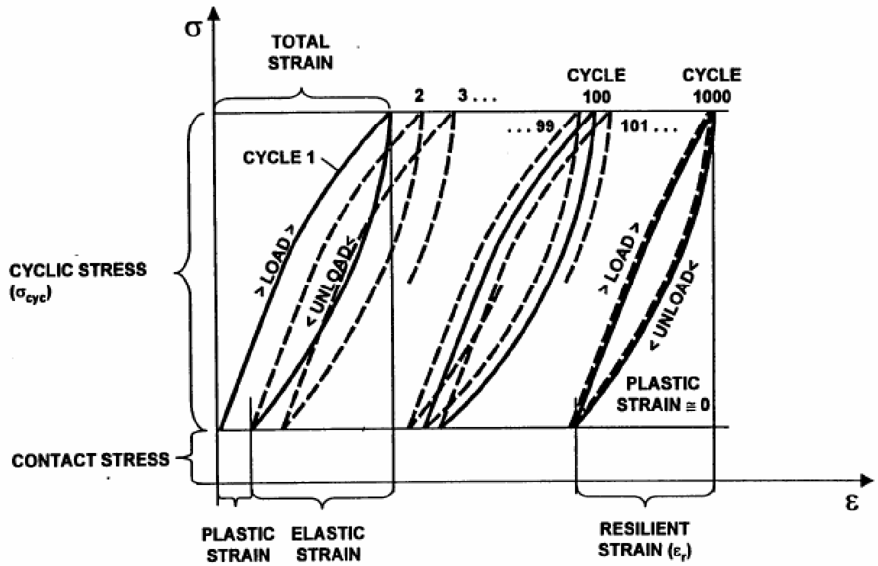

Figure 2.6 – Resilient Modulus (Puu Han, Y., (2005)).

From Figure 2.6 it is observed that the larger the number of cycles, the larger the permanent strain. However at a given cyclic deviator stress, which must be less than the one associated with failure (Section 3.3), the incremental plastic strain decreases with the number of load cycles.

The number of expected cycles in pavements is associated with the number of vehicles traveling on the pavement during the life of the road structure. This number varies drastically from less than a million of vehicles for small roads to tens of millions for busy Interstate highways.

The main characteristic of the resilient modulus is its stress-strain dependence, which means that its values changes as stress and strain conditions change (See Figure 2.7).

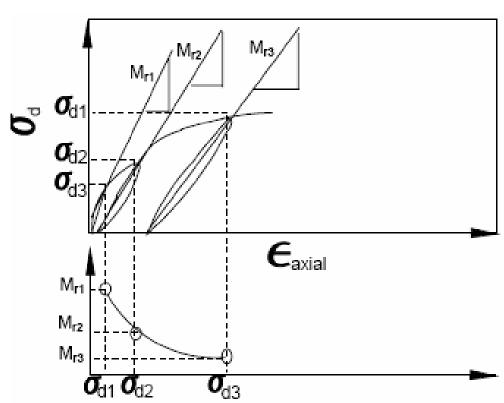

Figure 2.7 – Stress-strain hysteresis loop and resilient modulus determination (Hopkins, T.C et al., (2001)).

The mechanical characterization of the materials in pavement design is fundamental in determining the performance of the structure. Brown (1996) reports results of a sensitivity analysis which reveals that a change in the resilient modulus of the granular layer in a realistic range, using non linear models, caused a variation of the tensile strain by up to 70%. These calculations involved bituminous layer thicknesses between 4 in (100 mm) and 10 in (250 mm) with resilient modulus values between 300 lb/in^2 (2 GPA) and 1200 lb/in^2 (8 GPa).

The resilient modulus can be obtained using the following techniques:

- Backcalculation from nondestructive test (NDT) methods. NDT methods are usually preferred for rehabilitation of existing pavement structures. The most popular NDT methods include: plate load tests, Dynaflect, Road Rater, Falling Weight Deflectometer (FWD), and Spectral Analysis of Surface Waves (SASW). The plate load test and FWD can be used to evaluate the resilient modulus of the subgrades beneath existing pavements. The main problems of these tests are that they are time consuming (especially the plate load test) and have high uncertainty associated to them. An important aspect to be considered when FWD or plate load test are performed directly on subgrade is the magnitude of load, which has to be much lower than when the test is done on pavement surface in order to simulate actual working stress conditions.

- Laboratory testing devices include the triaxial test, resonant column test, simple shear device, torsional apparatus, hollow cylinder cell true triaxial cell, and the University of Illinois FastCell (which allows applying different stress paths). Because of its simplicity, the triaxial test is the most used and is part of the AASHTO T 297 standard resilient modulus determination that considers magnitudes and frequency of loading representative of working stress conditions (Section 2.3). However the estimated M_R using the triaxial test does not consider the rotation of principal stresses associated to the cyclic loads on pavements, shear stress reversal, nor the anisotropic nature of M_R. Additionally the lateral boundary conditions are different from insitu conditions.

- Correlations with soil properties and simple tests such as CBR, unconfined compressive strength, Resistant value (Ratio of the applied vertical stress to the developed horizontal stress) are tests mainly used in pavement material characterization. A fundamental problem with empirical relationships is that these formulas attempt to assign a fixed value of resilient modulus to a given type of soil. However the resilient modulus is a stress-strain dependent parameter.

The resilient modulus has provided a better way to understand and describe the pavement behavior, but has also brought some new problems, which in part are related to the resilient modulus lab test procedure. These problems include:

- Cost and complexity of the test procedure. These factors lead to performing only a few tests that makes it impossible to represent the subgrade and pavement materials under conditions than can be expected in the natural environment for a long period (e.g. soaked and un-soaked conditions, freeze-thaw, etc.).

- The time it takes for testing and for the operator to gain the required experience to perform a successful test.

- Difficulty of representing several field conditions. On this topic Hopkins, T.C., et al. (2001), who performed resilient modulus tests on un-soaked and soaked samples, reported that many soaked specimens "bulged" during repeated loading due to large excess pore pressures built up during testing. They suggested that more research is needed to determine the best approach to test saturated or nearly saturated clayey soils.

  On the other hand, specimens of base and subbase are prepared in the laboratory using procedures that are different from what the soils are subjected to in the field, which leads to testing materials with different structures than the one existing in the field.

- It does not provide the information needed during construction to determine whether the resilient modulus of the subgrade, subbase and base meet the design requirements.

Here are the main aspects of resilient modulus of clayey, sandy and silty-sand soils.

2.4.1 Resilient modulus of cohesive soils

Compaction method, matrix suction, water content, and compaction energy are the main factors that affect the resilient modulus (M_R) of cohesive soils. Here is a brief discussion of these factors.

- Method of compaction has been found to affect M_R. In general samples compacted statically display higher M_R compared to those created by kneading compaction, which is associated to a more dispersive structure.

- A linear relationship exists between the M_R and soil moisture suction. It has been proposed that the resilient modulus is a function of three stress variables: the net confining stress (σ_3-u_a), the axial stress (σ_1-σ_3), and the matrix suction (u_a-u_w), where u_a and u_w are the pore air and water pressure, respectively.

- For a given material, compaction parameters (moisture content and dry unit weight) control the modulus values. Dry side compaction leads to high M_R values. Resilient strains increase rapidly with increasing moisture content above the optimum value, causing a decrease in M_R. This is because at low water contents the water binds the particles and increases the effective stress between the particles through pore water suction and surface tension (See Figure 2.8).

- Figure 2.8 shows that cohesive soils compacted with high compactive effort exhibit a high M_R on the dry side of optimum, while the trend reverses at moisture contents wet from the optimum. Lee et al., (1997) observed the same trend for the unconfined strength at 1% strain.

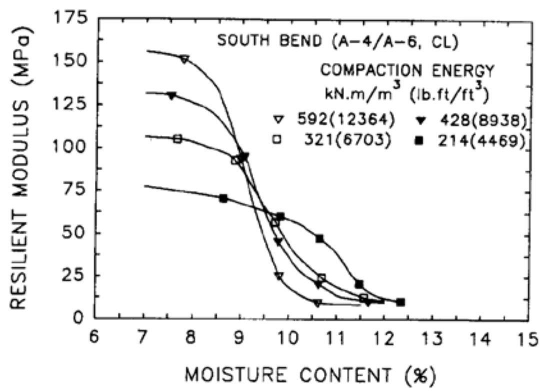

Figure 2.8 – Variation of M_R with water content, considering different compaction energies, using a low plastic clayey (CL) soil with optimum water content around 9.6%. The liquid limit and plasticity index are 23 and 10, respectively (Lee, W., et al., (1997)).

2.4.2 Resilient modulus of sandy soils

The most important factors affecting the resilient modulus (M_R) in sands are the state of stress, degree of saturation, initial density, and gradation.

Here is a brief discussion of the important compaction aspects of fine sands.

Dry unit weight – water content compaction curves

Sands compacted by impact tend to display straight line compaction curves throughout the range from the dry to the saturated condition. Additionally, vibratory compaction curves have been shown to be similar to impact compaction curves up to certain water contents that can be around 10%, after which vibratory methods lead to higher dry unit weights. During vibration, for water content around 12%, water is usually observed squeezing out of the mold during compaction. This is called the “flushed” or “wet as possible” compaction condition (See Figure 2.9).

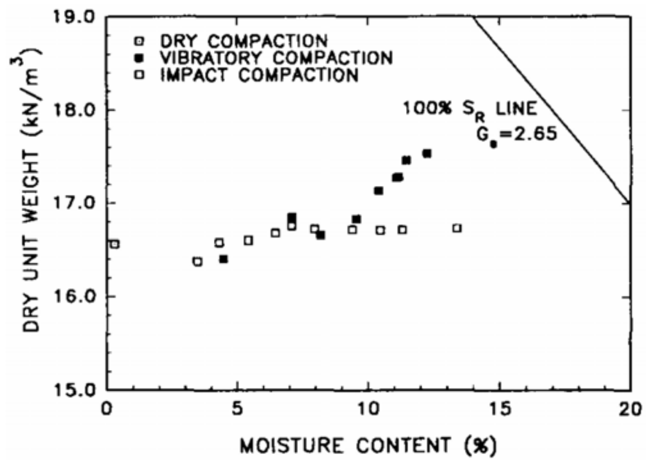

Figure 2.9 – Compaction test on fine sand using impact and vibration method (Lee, W., et al.., (1995))

Effect of compaction method on M_R

Vibratory-compacted specimens produce significantly less permanent deformation and moderately larger resilient moduli than impact-compacted specimens (See Figure 2.10). Lee, et al., (1996) found that the permanent strains of the impact-compacted specimens are about 2.5 times larger than that of vibratory-compacted specimens, and resilient strains of the impact-compacted specimens are about 20% to 40% larger than that of vibratory specimens.

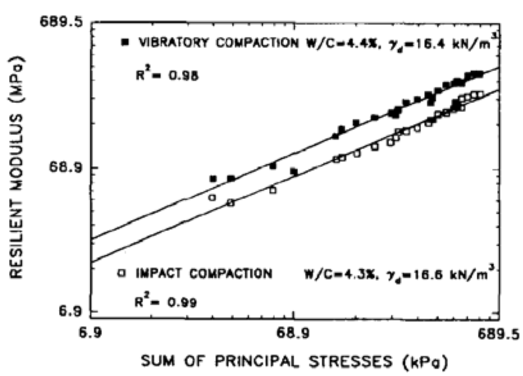

Figure 2.10 – Resilient characteristics of vibratory-compacted and impact-compacted dune sand (Lee, W., et al., 1995).

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The difference in behavior may be due to non-uniform densification of impact-compacted specimens because of the compaction effect. Specimens compacted by vibration are densified uniformly by means of surcharge at the top of the soil during compaction; specimens compacted by impact are densified less uniformly due to disturbance in the upper portion of each compacted layer. As a result, smaller reversible and the larger permanent deformations occur in less-densified layers when stress pulses are applied.

Effect of water content and dry density

The major effect of water content is to increase the permanent deformation of the specimen. The greater the water content the greater the permanent deformation. This was observed for vibratory and impact compaction by Lee et al., (1995).

On the other hand, increased dry density reduces the permanent deformation and increases the linear resilient modulus at relative compaction ranging from 95% to 103%.

Gradation

The resilient modulus is affected by the gradation curve, particle shape, and fines content. It has been reported that M_R decreases as the maximum aggregate size increase. This is important as the current standard resilient modulus test on granular materials has been conducted with maximum particle size of 0.75 in (19 mm). It is important to consider that removing particles greater than the specified size induces a change in the gradation of the material, and thus the material properties. In addition to these findings, it has been reported that M_R from 16 in (400 mm) diameter specimens differ from those of standard 6 in (150 mm) diameter specimens by a factor of 1.5 (Janoo, et al., 2004).

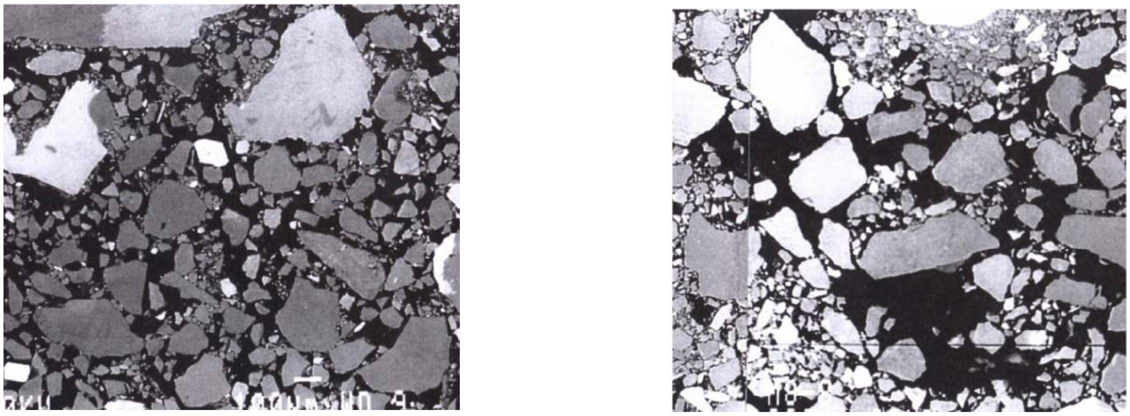
The effect of the angularity of the particles has been studied, and it has been reported that at low stress levels the M_R of river gravel is higher than that of crushed material. However, the trend is reversed at high stress levels (Janoo, et al, (2004)).

On the other hand, with regard to fine content, Janoo et al., (2004) reports that base course materials with more than 3% fines are prone to strength loss upon thawing.

2.4.3 Microstructure of non-plastic silty-sand soils

Compaction methods and gradation are the most important factors controlling the behavior of compacted granular materials. However, the presence of silts is important and the effect of water content becomes a decisive parameter in the determination of the microstructure.

Davoudi, M.H., et al (2005) studied the effect of compaction water content on the structure of a well-graded silty sand, containing 57% sand, 41% silt and 2% colloidal particles by means of scanning electron microscopy, water retention curve, permeability and mercury intrusion porosimetry test. Figure 2.11 shows microphotographs of specimens compacted at the dry and wet side of the optimum water content with almost identical porosity (26.7% and 26.2%, respectively).



a **b**

Figure 2.11 – Microphotographs of specimens compacted at different water content at about the same porosity. a) w = 4.76%, n = 26.7%; b) w = 8.71%, n = 26.2% (Davoudi, M.H. et al., (2005)).

In the specimen compacted dry of optimum (See Figure 2.11a) the silt particles are well distributed between the sand grains, providing a more or less uniform skeleton. This arrangement results in a porous system composed of relatively uniform macropores and micropores. Thus, compaction water content less than the optimum leads to increase permeability and stiffness of non-plastic soils.

On the other hand, the specimen compacted wet of optimum (See Figure 2.11b) is associated with an arrangement with a few large macropores and several smaller macropores. Figure 2.11b shows that the silt particles gather around the sand grains and form coarse aggregates, which can be easily deformed and pushed into large inter-aggregate pores under applied pressure. A large proportion of the porous volume is composed of micropores surrounding the fewer large macropores resulting in either a considerable degree of tortuosity within the macropore system which prevents macropores from fully contributing to the water flow (Davoudi, M.H, et al., (2005)). Thus, compaction water content greater than the optimum leads to reduce permeability and stiffness of non-plastic soils.

2.5 References

AASHTO T 292-97, (2000). Resilient modulus of subgrade soils and untreated base/subbase material.

Anguas, P.G, Gómez, J.A., (2001). Deformaciones permanentes en materiales granulares para la selección estructural de carreteras, Publicación Técnica No. 176 Sanfandila, Qro. (Spanish).

ARA, Inc., ERES Consultants Division, (2004). Guide for mechanical-empirical design of new and rehabilitated pavement structures www.trb.org/mepdg.

Briaud, J.L, and Seo, Jeongbok, (2003). Intelligent compaction: overview and research needs, Texas A&M University.

Brown, S.F., (1996). Soil mechanics in pavement engineering, The 36th Rankine Lecture of the British Geotechnical Society, Geotechnique 46, No. 3, 383-426.

Davoudi, M.H. and Lefebvre, G., (2005). Influence of compaction condition on the microstructure of non-plastic glacial till. 16th International Conference on Soil Mechanics and Geotechnical Engineering, Osaka, Japan.

Han, Y.P., (2005). Resilient modulus estimation system. PhD dissertation, University of Missouri-Rolla.

Hopkins, T.C, Beckham, T.L., Sun, C., Ni, B., (2001). Resilient modulus of Kentucky soils. Research Report KTC-01-07/SPR-163-95-1F.

Huang, Y.H., (1993). Pavement analysis and design. Prentice Hall, Englewood Cliffs, New Jersey.

Janoo, V, Bayer, J. and Benda, C., (2004). Effect of aggregate angularity on base material properties, Journal of Materials in Civil Engineering, Vol. 16, No.6.

Mohammad, L.N., Titi, H.H. and Herath, A., (2002). Effect of moisture content and dry unit weight on the resilient modulus of subgrade soils predicted by cone penetration test. Louisiana Department of Transportation and Development.

Lee, W., Bohra, N.C., and Altschaeffl, A.G., (1995). Resilient characteristics of dune sand, Journal of Transportation Engineering, ASCE, Vol. 121, No. 6, pp. 502-506.

Lee, W., Bohra, N.C., Altschaeffl, A.G., and White, T.D., (1997). Resilient modulus of cohesive soils, Journal of Geotechnical and Geoenvironmental Engineering, ASCE, Vol. 123, No. 2, pp. 131-136.

Seyhan, U. and Tutumluer, E., (2000). Advanced characterization of granular materials for mechanistic based pavement design, Pavement subgrade, unbound materials, and nondestructive testing. Edited by Michael S. Mamlouk. ASCE, Special publication, No 98, Denver, Colorado, pp. 51-72.

CHAPTER 3 - MODULUS IN PAVEMENT DESIGN

3.1 Introduction

The resilient modulus (M_R) was developed as part of the evolution of pavement design from methods based only on experience to methods based on the combination of mechanics theory, and field observations.

In the previous chapter, the main limitations of M_R laboratory test for resilient modulus determination were explored. The main drawbacks are: time consuming procedures, difficulty in modeling field loading conditions and environmental conditions (e.g. variation of saturation with time), and difficulty in the reconstruction of field soil structure. As a result these limitations, laboratory test data poorly represent the field compaction control process because the real conditions of the material in the field are not accurately modeled in the lab.

To overcome these problems and provide an acceptable option other than performing lab test, many researchers have concentrated their effort on the development of field tests and on correlating resilient modulus to soil properties and simple tests that are cheaper and more available in laboratories than the resilient modulus test.

In the development of field resilient modulus tests, the main issue is related to the determination of a representative modulus based not only on the characteristics of the materials but also on the boundary conditions (e.g. bed rock position, relative stiffness of subsoil layers), and loading (field working stress) conditions. On the other hand, when using correlations with simple tests, the main concern is related with how to relate the results from these simpler tests with a resilient modulus considering that the design philosophies used by these test methods are not exactly the same as for the resilient modulus test.

In this chapter we present a brief description of the conventional field tests used for compaction quality control procedures, pointing out their main advantages and limitations. Included are: the Falling-Weight Deflectometer (FWD), Spectral Analysis of Surface Waves (SASW), Stiffness Gauge, Plate Load Test, Dynamic Cone Penetration Test (DCP), and water content measurement tests. Most of these tests are classified as a Non-Destructive Tests (NDT). Considering that Indiana Department of Transportation extensively uses FWD, and the potential efficacy of this test for calibration of intelligent compaction technologies, this chapter presented presents a more detailed description of this field test.

The objective of this Chapter is to formulate test procedures for the application of a robust integrated quality control process involving conventional tests and advanced compaction technology as Intelligent Compaction (See Chapter 4 and 5). The interaction of these tools can be either in the formulation of correlations between them or in the verification of results on some selected spot locations in the field.

3.2 Falling Weight Deflectometer

Non-Destructive Tests (NDT) can be divided in two main categories; surface deflection-base basin methods and stress wave propagation methods.

In this section we discuss the first category, which is the most used, mainly due to the speed of operation. Stress wave propagation methods are discussed in Section 3.3.

In surface deflection basin tests, pavement structure and layer moduli are interpreted from the load-deformation response of the pavement system. The Falling-Weight Deflectometer (FWD) test is the main device representative of surface loading test (ASTM D4695-03 and ASTM 4694-96)

Several researchers made attempts at provide stable relationships between FWD test results and the Laboratory Resilient Modulus tests. William et al. (2000) reported that the ratio of lab values to the corresponding back-calculated values from FWD tests on pavement surface varied from 0.18 to 2.44. Newcomb (1987) reported the results of similar tests for the State of Washington with the ratio in the range of 0.8 to 1.3. Von Quintus and Killigsworth (1998) reported ratios in the range of 0.1 to 3.5, based on data obtained from the long-term pavement performance (LTPP) database. The AASHTO Guide (1993) recommends to using a factor of 0.33 to relate resilient modulus from FWD tests carried out on pavement surface, with resilient modulus from lab tests.

Comparison between lab resilient modulus tests and FWD tests performed directly on subgrade differ with the differences exhibiting both proportional and random behavior. The main causes are the inherent disturbance from sampling, difference in the tested soil volume, and different boundary conditions. Thus, equivalence of results between them should not be expected.

3.2.1 Equipment

In this section, we discuss the FWD equipment, the interaction of the loading FWD devices with the ground, and the methods for the estimation of modulus.

The FWD is a non-destructive test that imparts a transient load to the subgrade or pavement surface. The duration and magnitude of the applied force is representative of the load pulse induced by moving vehicles and aircraft wheel loads (See Section 2.2).

The FWD impulse test load is created by dropping weights from various heights onto a set of rubber buffers, mounted on a foot circular plate with a diameter of 12 inches (300 mm) (See figures 3.1 and 3.2). The applied force ranges between 1,500 and 25,000 lbf (7 and 110 kN) and is measured by a load cell. The resulting surface deflection is measured by a series of four to nine vertical velocity measuring sensors positioned along the ground surface at pre-determined intervals, (e.g. 12 inches (300mm)) from the loading plate (See figure 3.2).

Figure 3.1 – Falling INDOT Falling Weight Deflectometer device.

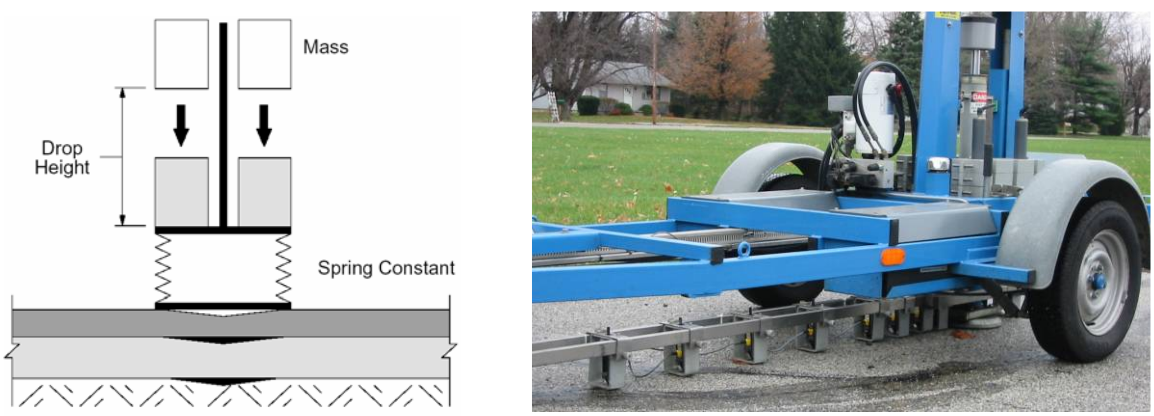


Figure 3.2 – FWD loading and sensor system
(Illinois Illinois DOT and http://www.wsdot.wa.gov/biz/mats/pavement/fwd.htm, INDOT, respectively)

Signals from the load cell and deflection velocity sensors are fed into the system processor, which filters and amplifies the signals used in the back-calculation of layer stiffness. The measured velocity is converted to vertical displacement. Basic linear elastic back-calculation methods select peak values and transfer this information to an onboard computer (See Section 3.2.4). The FWD test is associated with relatively low frequency components (High frequency is related to the surface wave method discussed in Section 3.3).

A computerized system in the tow vehicle monitors and controls the testing cycle. A typical test sequence is approximately one minute long, so testing proceeds rapidly down a street, highway, or airfield.

3.2.2 Loading plate-pavement interaction

The FWD test for compaction quality control purposes is carried out directly on the subgrade. However, the data tend to be less precise compared with tests on a pavement surface because of the rough surface, compared with a completed pavement, which can interfere with the geophones (Brown, 1996). Therefore, the subgrade surface profile being impacted affects the pressure distribution. This problem also exists for irregular and/or cracked pavement surfaces.

In order to have a better idea of the limitation of FWD tests conducted on subgrade, the main aspects of ground-FWD test interaction, and its effect on the measured response is presented in this section.

In the back-calculation of modulus using FWD deflections, a uniform distribution of load by the plate to the soil or pavement is assumed. To meet this requirement, manufacturers utilize a composite loading plate consisting of a steel plate, bonded to a PVC plate, and with a rubber pad placed on the lower face of the PVC plate. The main function of the rubber is to transform the semi-rigid loading plate to a flexible loading plate; thus producing uniform pressure distribution over the pavement (See Figure 3.2).

In addition, due to surface irregularities some manufacturers fabricate segmented load plates to distribute the load more evenly (Bush et al., Editors, 1989, Crovetti et al, 2000)). The selection of this configuration may be required when FWD is carried out directly on subgrades or pavement bases.

Boddapati et al (1989) studied the effect of stiffness and thickness of the loading plate components and pavement structural elements on the pressure distribution through an elasto-static FE model.

With regard to the pavement structure, Boddapati, et al. (2000) identified that as the stiffness of the pavement material decreases, the distribution of stress tends toward a non-uniform configuration. Thus, it is very difficult to get a uniform pressure distribution when the FWD is performed on thin asphaltic layers, pavement bases, and subgrades.

With regard to the loading plate components, Boddapati et al (2000) had the following observations:

- The influence of the plate stiffness on the stress distribution is small.
- The plate thickness considerably influences the central deflection magnitude.
- The thicker the plate the more non-uniform the pressure distribution.
- The stiffness and thickness of the PVC plate have a small influence on the pressure distribution.
- The rubber facilitates the distribution of the load to the surface. The exclusion of the rubber results in significant concentration of stress at the outer edge of the plate, inducing a yielding stress state.
- The stiffness of the rubber controls the stress distribution. The softer the rubber the more uniform the stress distribution at the loading plate-surface interface contact.
- The deviation of the stress distribution from uniformity affects the deflection measured at the center of the loading plate. The deflections measured by the adjacent sensors are typically not affected.

Having a non-uniform pressure distribution, considerably affects the backcalculation of the moduli. Crovetti et al (1994) suggested that direct measurements of the applied pressure distribution under FWD load plate be made to characterize the distribution that would facilitate the refinement of the analysis of modulus estimation. Crovetti et al (1994) used pressure sensitive film manufactured by Fuji Film I&I, sensitive in the range from 70 to lb/in^2 (480 to 2400 kPa).

More recently Roche et al. 2004 compared the use of a plate consisting of two semi-circular plates with a rigid plate as described above. They concluded that "the falling weight deflectometers equipped with split plates impart more uniform load to the pavement. The split plate in general improved the performance of the FWD. However, the deflections measured with the two plates are different. As such, should TxDOT decide to utilize split load plates, a means of adjusting the deflections measured with the new configuration to those historically measured with the solid plate should be devised."

3.2.3 Repeatability and Reproducibility

In practice different FWD equipment are used to determine moduli in road projects. However, the results may differ from one device to another.

Van Gurp (1991) evaluated the repeatability and reproducibility of commercially and so-called "home-made" FWD devices. Repeatability is the production of consistent results in one spot. Reproducibility assessment among FWD devices implies the study of the type of relationship between results and equipment.

The exact procedure for the determination of the peak values of load and deflections by the electronic processing units of the FWD signals is manufacturer dependent. Differences in operation modes, rubber pads under the loading plates and sensors, properties and number of rubber buffers mounted, drop weight and drop height, pulse shape and pulse width, can all lead to test results different from apparently similar devices.

The above mentioned items require careful analysis of testing equipment procedures, testing procedures, and back-calculation techniques before comparing information from different equipment.

The main findings by Van Gurp (1991) related with consistence of FWD results are:

- The manufacturers of FWDs present an acceptable and higher repeatability than the one associated with the so called "home-made" devices. Variation in FWD load data, in a series of multiple drops, is usually confined to two percent of population mean.
- Calibrated devices can present larger differences in deflection peak values mainly due to differences in loading times (rise time) and pulses energies. Thus, an FWD may record different peak deflection values when either pulse width, or rise time, or pulse shape are different. In order to obtained reproducible data, pulse energies must be specified.

3.2.4 Back-calculation methods

Back-calculation procedures are separated in five categories:

- Static linear elastic
- Static nonlinear elastic
- Dynamic linear using frequency domain fitting
- Dynamic linear using time domain fitting
- Dynamic nonlinear analysis.

Advanced methods require detailed material characterization models.

Different moduli are usually obtained from different back-calculation computer programs, indicating that the calculation process is very sensitive to the type of analysis and assumptions underlying the analysis.

Numerous researchers have documented the incompatibility between dynamic FWD testing and back-calculation with static analyses. The effect that the dynamic nature of the FWD test has on the deflection is significantly influenced by the material and geometric configuration of the pavement system, particularly the depth to bedrock and the subgrade modulus.

A flexible pavement with a shallow bedrock (e.g. less than 9 ft (3 m)) and a soft subgrade exhibits a fundamental frequency in the vicinity of the 30 Hz (calculated from $V_s/(4H)$) and thus may go under dynamic amplification with FWD loading (Mooney et al, 2000). Static back-calculation does not consider these dynamic factors and thus, in the presence of a shallow stiff layer and a soft subgrade, the statically back-calculated subgrade moduli can be considerably underestimated. On the other hand, in the presence of a deep rigid layer, amplification due to resonance does not occur, which may decrease surface deflections.

Therefore, back-calculation of resilient modulus from FWD test results must consider bedrock location. Chang (1991) indicated that only the free vibration part of the FWD displacement-time history may be correlated to the depth to bedrock. Seng (1993) found that the depth to bedrock could be related to the frequency of the free vibration portion of FWD sensors and the shear wave velocity in the subgrade layer. The major shortcoming in Seng's approach is that the free vibration amplitudes from FWD sensors are extremely small, which makes accurate measurements of the free vibration period difficult if not impossible for many sites (Mooney et al, 2000).

All back-calculation procedures use error minimization techniques to minimize either the absolute or the square error, with or without using weight factors. A description of the methods is beyond the scope of this synthesis study.

3.2.5 FWD on subgrade

Based on the discussion of loading plate-ground interaction presented in Section 3.3.2, the following aspects have to be considered in the evaluation of results from FWD test performed directly on base and subgrade:

- Due to the marked difference between the stiffness of the ground and the loading plate the estimated modulus from deflection basins with the FWD is less precise due to the non uniform stress distribution under the loading plate. Additionally, the rough surface can interfere with the geophones (Brown, 1996).

- In Section 3.3.2 it was mentioned that the stiffness of the rubber controls the stress distribution. The softer the rubber the more uniform the stress distribution at the loading plate-surface interface contact. This aspect supports the need for research in the application of FWD on soils, where a much softer rubber may be required than that the one regularly used.

- Sensor readings can be affected by plastic deformation, especially under the loading plate. To ameliorate the problem, four conditioning drops usually are indented used for reducing plastic deformation.

- Another aspect to consider is the confinement resulting from the overlying pavement structure and dissipation of stress through the pavement structure. The back-calculated subgrade modulus from FWD tests performed on the pavement surface compared to the subgrade modulus back-calculated from FWD test carried out directly on the subgrade increase by 40% for fine-grained soils, whereas the increase is 100% for coarse-grained soils.

- Also, the load applied to a subgrade by the FWD should be much smaller than the load applied by the FWD to the surface of the pavement to account for the reduction in stress by the pavement structure.

- Another approach that should be considered is the response of the FWD loading plate alone and not use the measurements of the transducers at various radii away from the plate. It would be similar to the static plate load test, but done dynamically. Analysis would make use of the impedance methods developed by Gazetas (1990) for the vibration of foundations.

3.3 Spectral analysis of surface waves (SASW)

3.3.1 Test description

Stress wave tests are based on the generation of stress waves at one point in the pavement structure and measuring the times required for the wave to propagate to other points on the pavement surface. The main tests in this category are:

- Spectral analysis of surface wave method (SASW). This method is mainly used to evaluate layer moduli of pavements, bases, and subgrades.
- Crosshole seismic method.
- The impact-echo test for measuring the thickness of concrete slabs (ASTM D4694-96)

The SASW has been under continuous development at the University of Texas at Austin since 1980, and at a number of other universities including: Georgia Tech, the University of Kentucky, University of Michigan, Penn State University, Rutgers University, and University of Texas at El Paso.

The SASW is based on the generation and detection of Rayleigh waves from the surface of pavements systems and subgrade.

The propagation of waves in soil is affected by the vertical heterogeneity of soils mainly represented by the increase of soil stiffness with depth due to confinement, which makes Rayleigh waves with long wavelength (low frequency waves) travel faster than the ones associated with short wavelengths (high frequency waves). This frequency dependence of the surface wave is called dispersion. The materials in which this phenomenon occurs are identified as dispersive.

Waves in soils exhibit a characteristic group velocity as well as a characteristic phase velocity. Phase velocities describe the rate at which points of constant phase (frequency) travel through a medium. Group velocity describes the travel rate of packet of waves containing a range of frequencies. In non-dispersive materials the group velocity is equal to the phase velocity. In dispersive media such as soils the group velocity is lower than the phase velocity. Thus, a wave packet would consist of a series of individual peaks that appear at the back end of the packet, move through the packet to the front, and disappear (Krammer, 2000).

The velocity of propagation of interest in SASW testing is surface wave phase velocity, sometimes identified as the apparent surface wave velocity, with which a seismic disturbance of a single frequency is propagated in the medium.

Even though the SASW method has a strong theoretical basis, it is not commonly used in the field practice because of the following reasons:

- This test is time consuming. However, the seismic pavement analyzer (SPA) is an automated version of the SASW test that was introduced as a result of an initiative by the Strategic Highway Research Program (SHRP), and in essence is a portable version of the seismic analysis of surface wave (SASW) introduced in the early 1980s. Details in SPA are presented in Nazarian, et al. (1993).
- The interpretation of the information requires significant experience, as well in the adjustment of field configuration.
- The stress level associated with the test is related with strains below 0.001%, which are well below the ones representative of pavement working load conditions. Loizos et al (2003) reports that good pavement and rail track performance is associated with small to intermediate strain levels, with amplitudes below 0.1%.

3.3.2 Test Procedure

The test requires a source, receivers, and an analyzing device. The sources have to be capable of generating waves from 10 Hz and to more than 50 kHz. Normally small and large hammers are used to generate high and low frequency waves, respectively. Nazarian et al (1989) reported the use of pulsating crystals for the generation of waves with frequencies higher than 20 kHz.

The record of wave arrival can be done with accelerometers and velocity transducer (geophones), which are secured to the surface. These capture high and low frequency waves, respectively. Hiltunen et al (1989) recommended Common Source Geometry receiver-source set up, which employs a fixed source location and places the receivers at appropriate distance away from the source to achieve the desired receiver spacing.

The primary goal of the SASW test is to obtain a dispersion curve, which represents the relationship between the phase velocity and the wavelengths. The calculation of this curve is based on two spectral functions that are measured in the field; the cross power spectrum (CPS) and the coherence function.

These spectral functions come from the spectral analysis of the Fast Fourier Transform (FFT) of the captured vibration transducer signals at each geophone.

The last step is an inversion process through which the modulus profile of the ground is estimated. Poisson's ratio and mass density are required input data in the analysis.

One critical stage of the SASW test is the selection of the seismic waves (Tawfiq, K. et al., (2002)). During field testing, only the time history records are usually monitored from the display. The testing crew often determines the reasonableness of the measurements from the shape of the displayed waveforms based on personal judgment. This task becomes cumbersome considering the multiple signals (direct arrival and reflected compression waves and shear waves in addition to the surface waves), associated with each of the accelerometers or geophones used in the test. The selection of improper signals leads to miscalculated dispersion curves and therefore, modulus.

Tawfiq, K. et al. (2002) suggested categorizing seismic records based on waveforms and a factor used to quantify the amount of irregularities in the signals. This factor can be used at the site during the field testing to eliminate the guessing process in the selecting the signals.

3.4 Stiffness Gauge

The Stiffness Gauge is a portable non-invasive electro-mechanical device that estimates stiffness by the application of steady-state sinusoidal vibratory loadings applied over a range of frequency from 100 to 200 Hz on the ground surface (See Figure 3.3).

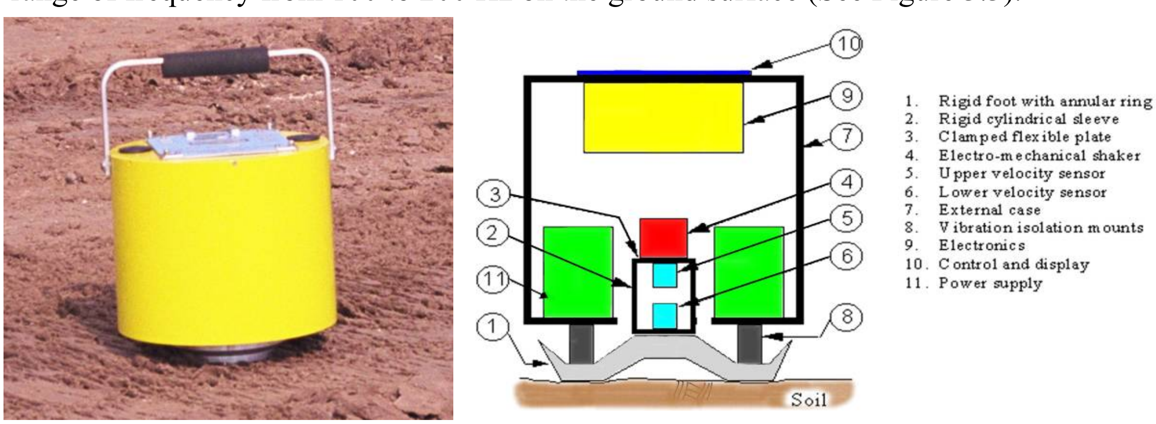


Fig. 3.3 Stiffness Gauge: a) Device in use; b) Schematic Diagram of Components (Ref., Fiedler, S., Humboldt Manufacturing, Inc. http://www.humboldtmfg.com)

The ASTM Standard D 6758-02 specified that the equipment has to weigh enough to produce a contact stresses between 430 lb/ft^2 (20.6 kPa) and 575 lb/ft^2 (27.6 kPa). (The diameter of a typical device is 11 in (28 cm).

The stiffness calculation is given by the average of the force applied by the shaker over the operating frequencies of the device. The test avoids the requirement of a non-moving reference for the ground displacement measurement by calculating the stiffness based on the measurement of the velocity of the flexible internal plate and that of a rigid footing to determine the dynamic force applied to the circular ring footing and the resulting displacement (ASTM D6758-02).

Use of the equipment requires that it be placed on a moist sand layer that provided provides a uniform contact between the rigid ring footing and the soil surface. The manufacturer advertises that the measured depth is between 9 in (230 mm) to 12 in (310 mm) from surface. The test needs to be run in a location relatively free of construction noise and vibration.

The shear modulus is estimated assuming a Poisson's ratio and using the elastic solution of an annular loading on the surface of a halfspace.

3.5 Plate load test

The plate load test is one of the first field tests that has been extensively used to overcome the limitations of lab tests due to sampling disturbance and scale effects associated with laboratory testing of small elements with boundary conditions that may not be representative of field conditions.

The loading and unloading plate test uses a relative small diameter loading plate, usually no greater than 2.5 ft (0.75m), where a load is applied against a heavy reaction (typically a truck or a piece of earthmoving equipment). A complete description of the test equipment and procedure is presented in the Standard ASTM D1195-93 (1997).

For pavements and soils, static plate loading tests have been used for many years, initially to determine "modulus of subgrade reaction" (k_s). The k_s is a conceptual relationship between soil pressure and deflection, and it is related to the elastic modulus by the geometry of the plate and the Poisson's ratio of the medium. The plate load test also is used as indirect technique for assessing the California Bearing Ratio.

The depth of influence of the test is directly related to the size of the loading plate. As the size increases, the load-displacement response tends to be stiffer due to stiffness increase with confinement with depth. Thus, k_s is loading area size dependent.

The static plate loading test also is cumbersome and time consuming. More importantly, it does not represent the pavement working loading conditions, for which the loading rate is fundamental (See Chapter 2). In particular, a static test on a material with a high degree of saturation can allow pore pressure dissipation and result in more favorable results than a transient load test which is essentially an undrained event (Brown, 1996). Thus, the use of the plate load test in pavements is constrained to cases of stationary loads, like the ones associated with the load of aircraft parked overnight (Hall, 1994).

Considering the time-consuming limitations and drainage conditions of this test, Briaud et al (2004) has developed the Briaud Compaction Device BCD test (Fig. 3.4) that estimates the ground elastic modulus using the bending of a plate resting on the surface when a quasi-static load is applied by the operator. The bending strain, measured with strain gauges glued to the top of the plate, is that existing over a few seconds after the application of the load.

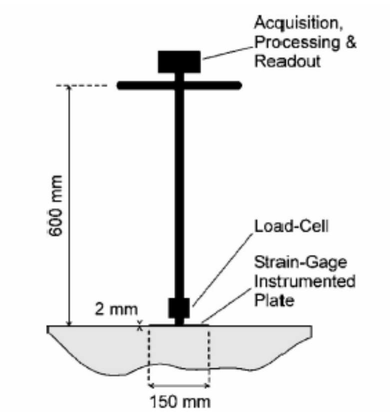


Fig. 3.4 Briaud Compaction Device: a) Schematic diagram; b) Use in the field to measure compaction; c) Use in a compaction mold. (Ref., Briaud, et al. (2006)).

This test can be performed in the lab, using compaction molds and in the field in a small fraction of time (less than a minute) that is associated with the plate test (an hour or more).

3.6 Dynamic Cone Penetration Test (DCP)

Historically, the use of DCP was initiated by Scala of Australia in the middle of 1950s who developed the Scala penetrometer for assessing the in-situ CBR for cohesive soils. The Scala device included a 20 lb (89 N) drop hammer falling a distance of 20 in (508 mm). A 5/8 inch (5.9 mm) diameter rod calibrated in 2 inch (50.8 mm) increments was used to determine the penetration. This configuration used a 30 degree included angle cone tip (George et al, 2000).

The next generation of DCP equipment was developed by Van Vuuren from South Africa. Basically it was similar to the DCP apparatus developed by Scala except the weight of the drop hammer was changed to 22 lbs (98 N) and the drop height was changed to 18.1 in (383.5 mm). The shaft diameter measured 0.63 in (16 mm) while the apex angle remained at 30 degrees.

The present version of the DCP was developed in South Africa as well. The hammer weight was reduced to 17.6 lbs (78 N) and the height of the drop was increased to 22.6 inches (576 mm). The cone tip angle is 60 degrees and its diameter is 0.79 inch (20 mm) (ASTM D6951-03).

The use of DCP has evolved from CBR correlation to modulus estimation. Nazarian et al. (2000) presented an instrumented DCP device for pavement characterization purposes. This equipment, still under development, estimates Poisson's ratio and modulus.

Herath, et al. (2005) explored the relationship of the DCP penetration index and resilient modulus laboratory test with water content, dry density, and Plasticity Index. Based on the statistical analysis of DCP test results performed in chambers and M_R lab test on compacted specimens of clay soils, Herath, et al. (2005) proposed the following two equations that related M_R and DCP test.

$$M_R = 16.28 + \frac{928.24}{(DCPI)} \tag{3.1}$$

$$M_R = 520.62\left(\frac{1}{(DCPI)^{0.7362}}\right) + 0.40\left(\frac{\gamma_d}{w}\right) + 0.44PI \tag{3.2}$$

where M_R is given in MPa, *DCPI* is the DCP penetration index (mm/blow), *PI* is the Plasticity Index (%), *w* is the water content (%), and γ_d is the dry unit weight (kN/m^3).

The correlations were validated with field DCP tests, lab DCP tests, and lab M_R tests that were not included in the derivation of the equations. The Plasticity Index of the tested clayey soil range between 4 and 26, the optimum water content was between 12% and 20%, and maximum dry unit was between 95 and 121 lb/ft^3 (15 kN/m^3 and 19 kN/m^3), respectively. Figure 3.5 presents the performance of the empirical equations.

3.7 Water content

Water content is one of the most critical factors in the compaction and construction of earth structures.

At the dry side of the compaction curve, clayey soils behavior is controlled by the matrix suction, while at the wet side, is controlled by the soil particle-water interaction represented by the double layer thickness. Section 2.4.1 discussed the effect of water content on resilient modulus.

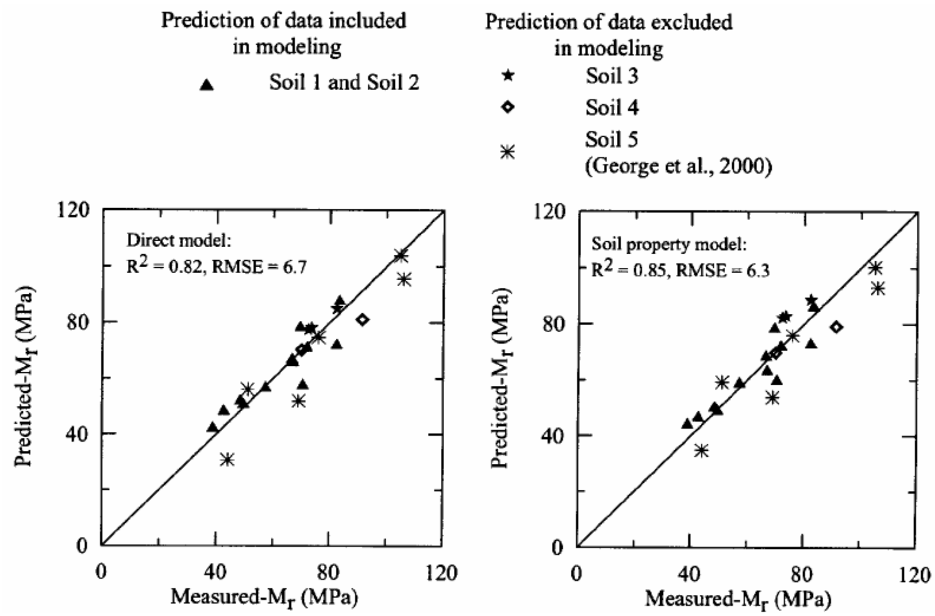


Figure 3.5 - Prediction of M_R for cohesive soils using Equations 3.1 (left chart) and 3.2 (right chart).

In sandy soils, a better readjustment of the particles is reached at high water contents, where vibratory compaction is more effective due to the lubrication effects of water.

Measurement of soil density and water content in compacted fills is the principal means of quality control in the US. The traditional testing methods include rubber balloon density apparatus, sand cone test, and nuclear density meter. The nuclear density meter estimates both water content and total density. However, the current method presents various limitations, including the use of hazardous materials, limitations in accuracy, and regulatory requirements.

TDR, a promising technology that uses electromagnetic waves to measure water content and dry density, is an option to overcome these limitations. Topp et al (1980) established a relation between soil volumetric water and soil apparent dielectric constant. Later research shows that, in additional to dielectric constant, it is also possible to obtain bulk electric conductivity from TDR wave forms (Dalton et al. 1984).

Considerable improvements have been achieved at Purdue University in the extension of the technology to geotechnical engineering application (Siddiqui et al. 1995, Feng et al. 1999, Lin, C.P., 1999, Drnevich et al. 2001a, 2002, Yu et al. 2004). An ASTM standard designated ASTM D6780-05 (2005) for the method developed at Purdue has been approved.

3.8 *References*

ASTM D1195-03, 1997. Standard test method for repetitive static plate load test of soils and flexible pavements components for use in evaluation and design of Airport and Highways Pavements.

ASTM D6758-02, 2002. Standard test method for Meassuring stiffness and apparent modulus of soil and soil – aggregate in place by and electro-mechanical method.

ASTM D6795-03, 2003. Standard guide for general pavement deflection measurements.

ASTM D6794-96, 2003. Standard test method for Deflections with Falling-Weight-Type Impulse Load Device.

ASTM D6951-03, 1998. Standard test method for Use of Dynamic Cone Penetrometer in Shallow Pavement Applications.

ASTM D6780-05, 2005. Standard test method for water content and density of soil in place by Time Domain Reflectometry (TDR).

Briaud, J.L., and Li, Y., and Rhee, K. (20042006). BCD: A Soil Modulus Device for Compaction Control,A BCD for compaction control. Journal of Geotechnical and Geoenvironmental Engineering, Vol. 132, No. 1, January 1, 2006, pp. 108-115.Proposed for publication in the Journal of Geotechnical and Geoenviromental Engineering, ASCE.

Boddapati, K.M. and Nazarian, S., 1993. Effect of pavement-falling weight deflectometer interaction on measured pavement response. Nondestructive Testing of Pavement and Backcalculation of moduli, V2. Edited by Von Quintas, H.L., Bush, A.J. and Baladi, G.Y. ASTM.

Brown, S.F., 1996. Soil mechanics in pavement engineering. Rankine Lecture. Geotechnique 46, No. 3, 383-426.

Crovetti, J.A., Shahin, M.Y. and Touma, B.E., 1989. Comparison of two falling weight deflectometer devices, Dynatest 8000 and KUAB 2M-FWD. Nondestructive Testing of Pavement and Backcalculation of moduli. Edited by Bush, A.I. and Baladi, G.Y. ASTM.

Crovetti, J.A. and Shahin, M.Y., 1993. The effect of annular load distribution on the backcalculated moduli of asphalt pavement layers. Nondestructive Testing of Pavement and Backcalculation of moduli, V2. Edited by Von Quintas, H.L., Bush, A.J. and Baladi, G.Y. ASTM.

Dalton, F.N., Herkelrath, W.N., Rawlins, D.S. and Rhoades, J.D., 1984. Time Domain Reflectometry: Simultaneous measurement of soil water content and electrical conductivity with a single probe. Science, 224(4652), 989-990.

Drnevich, V.P., Lin, C.P., Yi, Q., Yu, X, and Lovell J., 2001a. Real time determination of soil type, water content and density using electromagnetics. Rep. No.:FHWA/IN/JTRP-2000-20, Joint Transportation Research Program, Indiana Department of Transportation-Purdue University.

Drnevich, V.P., Yu, X., Lovell, J., and Tishmack, J.K., 2001b. Temperature effects on dielectric constant determined by Time Domain Reflectometry. TDR 2001: Innovative applications of TDR technology, Infrastructure Technology Institute, Northwestern University, Evanston, I11.

Drnevich, V.P., Yu, X. and Lovell, J., 2002. A new method for water content and in situ density determination. Proc., Great Leaks Geotechnical and Geoenvironmental Conference, Toledo, Ohio.

Feng, W., Lin, C.P., Dechamps, R.J., and Drnevich, V.P., 1999. Theoretical model of multisection time domain reflectometry measurement system. Water Resources Research, 35(8), 2321-2331.

Gazetas, G., (1990), Chap. 15 *Foundation Vibrations,* Foundation Engineering Handbook, Fang, H-Y, Second Edition, Van Nostrand Reinhold, NY, pp. 553-593.

George, K.P, and Uddin, W. (2002) Subgrade characterization for highway pavement design. Final report conducted by the Department of Civil Engineering, The university University of Mississippi.

Hall, J.W. and McCaffrey, P.S., 1993. Misleading results from nondestructive testing – a case study. Nondestructive Testing of Pavement and Backcalculation of moduli, V2. Edited by Von Quintas, H.L., Bush, A.J. and Baladi, G.Y. ASTM.

Herath, A., Mohammad, L.N., Gaspard, K., Gudishala, R. and Abu-Farsakh, M.Y. (2005). The use of dynamic cone penetrometer to predict resilient modulus of subgrade soils. Geotehcnical Geotechnical special Special publications Publications 130-142 and GRI_18, January 24-26, 2005, Austin Texas. Edited by Ellen M. Rathje. ASCE.

Hiltunen, D.R. and Woods, R.D., 1989. Influence of source and receiver geometry on the testing of pavements by the surface wave's method. Nondestructive testing of pavements and backcalculation of moduli. Albert J. Bush III and Gilbert Y. Baladi editors, ASTM-SPT 1026.

Hiltunen, D.R. and Woods, R.D., 1989. Influence of source and receiver geometry on the testing of pavements by the surface wave's method. Nondestructive testing of pavements and backcalculation of moduli. Albert J. Bush III and Gilbert Y. Baladi editors, ASTM-SPT 1026.

Kramer, S.L., 1996. Geotechnical earthquake engineering. Prentice-Hall.

Lin, C.P., 1999. Time domain reflectometry for soil properties. PhD thesis, School of Civil Engineering, Purdue University, West Lafayette, Ind.

Loizos, A. Boukovalas, G. ad Karlaftis, A., 2003. Dynamic stiffness modulus from pavement subgrade evaluation. Journal of transportation engineering, Vol.129, No4, July, 1, ASCE.

Livneth, M. Livneth, N.A. and Ishai, I. (2002). The Israeli experience with the regular and extended Dynamic Cone Penetrometer for Pavement and Subsoil-strength Evaluation. Nondestructive testing of Pavement and backcalculation of moduli. Vol. 3. ASTM, Shiraz D. Tayabji and Erland O. Lukanen Editors.

Mahoney, J.P, 1999. Pavement testing and backcalculation technologies - Challenges for the 21st century. Nondestructive Testing of Pavement and Backcalculation of moduli, V3. Edited by Tayabji, S.D. and Lukanen, E.O. ASTM.

Mooney, M.A., Bong, W, and Miller, G., 2000. The Determination of soft subgrade modulus for airport pavement rehabilitation via backcalculation of falling weight deflectomenter.

Nazarian, S. and Stokoe II, K.H., 1989. Nondestructive evaluation of pavements by surface wave method. Nondestructive testing of pavements and backcalculation of moduli. Albert J. Bush III and Gilbert Y. Baladi editors, ASTM-SPT 1026.

Nazarian, S. and Desai, M. (1993). Automated surface wave testing: Field testing, J. Geotechnical Engineering, 119(7), 1094-111.

Nazarian, S., Tandon, V., Crain, K. and Yuan, D. 2000. Use of instrumented Dynamic Cone Penetrometer in pavement characterization. Nondestructive testing of Pavement and backcalculation of moduli. Vol 3. ASTM, Shiraz D. Tayabji and Erland O. Lukanen Editors.

Rahim, A. and George, K.P., 2003. Falling weight deflectometer for estimating subgrade elastic moduli. Journal of Transportation Engineering, Vol 129, No. 1, January 1. ASCE.

Rocha, S., Nazarian, S., and Tandon, V. 2004. Impact of Solid and Split Plates on Response of Falling Weight Deflectometer, Final Report for Research Project 1784-3 Evaluate Reproducibility of TxDOT FWD Fleet and Recommend Improvements to FWD Calibration Procedure, Texas Department of Transportation, Center for Transportation Infrastructure Systems, The University of Texas at El Paso, El Paso, TX 79968-0516, March

Rodriguez, J., Ferregut, C. and Nazairan, S., 1992. Impact of variability in pavement parameter on backcalculated moduli. Road and airport pavement response monitoring systems, Edited by Vincent C. Janoo and Robert A. Eaton, ASCE.

Russuell, H.S., Hossain, M., 2000. Design resilient modulus of subgrade soils from FWD test.

Siddiqui, S.I., and Drnevich, V.P., 1995. A new method of measuring density and moisture content of soil using technique of Time domain reflectometry. Rep. No.: FHWA/IN/JTRP-95/9, Joint Transportation Research Program, Indiana Department of Transportation, Purdue University.

Siddiqui, S.I., Drnevich, V.P. and Dechamps, R.J. , 2000. Time domain reflectometry development for use in geotechnical engineering. Geotech. Test. J., 23(1), 9-20.

Tawfiq, K., Armaghani, J. and Sobanjo, J. (2002). Rational method for selecting seismic waves for pavement evaluation, Journal of Transportation Engineering, Vol. 128, No. 6, November 1, 2002, ASCE.

Topp, G.C., Davis, J.L., and Annan, A.P., 1980. Electromagnetic determination of soil water content: Measurements in coaxial transmission lines. Water Resources Research, 6(3), 574-582.

Uzan, J. 1993. Advanced backcalculation Backcalculation techniquesTechniques. Nondestructive Testing of Pavement and Backcalculation of moduliModuli, V2. Edited by Von Quintas, H.L., Bush, A.J. and Baladi, G.Y., ASTM.

Van Gurp, C. 1991. Consistency and reproducibility of falling weight deflections, Edited by Vincent C. Janoo and Robert A. Eaton, ASCE.

William, G.W. and Shoukry, S.N., 2000. Mechanistic evaluation of the depth to bedrock and its effect on the response of 3D FE pavement models.

CHAPTER 4 - ADVANCE COMPACTION TECHNOLOGY

4.1 Introduction

Soil compaction was a trial-error process until the 1930s, when R.R. Proctor identified the factors that control the soil microstructure in the compaction: the soil type, the water content at compaction, and the amount of compaction energy imparted to the soil (White et al., 2004). Subsequent research identified that the compaction method is also an important factor (Bell, J.R., 1977; Seed et al., 1959).

Since then, the conventional standard for compaction quality control procedure mainly has been based on:

- Process control: This factor is associated with the compaction energy imparted to the soil. In this stage lift thickness and number of passes of the compaction equipment are evaluated.

- Post construction spot test: In this stage the determination of dry density and water content at the end of the compaction is performed using spot tests like sand cone and water balloon test, for dry density; and nuclear gauge and TDR for both water content and dry density estimation. However, relying only on spot tests is an unreliable way to represent the compaction condition of the entire worked area considering that the volume of soil being tested is only about 1000 cm^3. Compaction standards of different countries require to take one sample per 2000 m^3 of compacted soil, which means a relation between sampled volume and compacted volume of 1:2000000 (Thurner, H. F et al., 2000).

In the USA, State Departments of Transportation and contractors have identified that the existing methods for measuring density are time consuming, labor-intensive, dangerous, and/or have uncertain accuracy. The previous characteristics have lead to two critical conditions (http://www.tfhrc.gov/pubrds/marapr98/soil.htm):

- Construction sites are often under sampled, causing inadequate compaction to go undetected or feedback to be provided too late for the cost-effective correction of problems.
- Designers tend to over-specify considering the high uncertainty and variability of the properties of the materials, and at the same time, contractors tend to over compact to ensure acceptance and avoid rework. All of these factors lead to the increment of construction time and cost.

The presented limitations suggest a change in the compaction Quality control/Quality Assurance (QC/QA) process to achieve the following targets:

- Reduction of uncertainty by increasing the sampling rate.
- High speed quality control procedures.

- Quality control techniques that are coupled to the construction process, allowing the availability of real-time processed data that let to guide de compaction work.

As discussed in Chapter 2, the design of pavements is in the transition process from an empirical to mechanistic-empirical approach, which will require a QC/QA procedure not only based on water content and dry density determination, but also on deformability and shear strength parameters that define the earth-structure behavior. Two soil specimens can have the same dry density but different resilient modulus and shear strength due to difference in their microstructure and water content.

Considering all of the above mentioned limitations and needs, advance compaction technologies have been in development for the last 25 years. This chapter introduces Continuous Compaction Control and Intelligent Compaction, which are quality control procedures totally integrated to the construction process that makes them capable of providing an 100% of coverage of the worked area and a real-time feedback.

4.2 Advanced Compaction Technology (ACT)

Increasing the demands for high quality construction at a low price and during a short period of time has lead to technology development in all construction areas, and earthwork compaction and compaction quality control are not the exception. Advanced Compaction Technology (ACT) is a phrase applied to all technologies that seek to automate and provide real-time quality control for compacting soils.

The development of ACT started in Europe at the end of the seventies with the continuous compaction control (CCC) method from vibratory compaction. CCC is a quality compaction method that allows having a QC/QA of 100% of the worked area using the compaction equipment as a measurement instrument of the dynamic roller-ground system response. For this purpose the roller is instrumented with accelerometers.

Further development of the ACT in Europe during the last 10 years led to the origin of the so called Intelligent Compaction (IC), which complements the CCC with a control system that uses the CCC collected information to continuously adapt the performance of the compaction equipment (i.e. roller speed, frequency and amplitude of roller motion and drum motion mode) to optimize compaction and meet the specified construction standards.

Most of the ACT research has been conducted by private research centers of equipment manufacturers. Detailed information of their research is not totally available for public access due to business concerns. The main European manufactures of IC are BOMAG from Germany and AMMANN from Switzerland. Geodynamic from Sweden is leading CCC technology and manufactures auxiliary equipment that can be easily installed in regular vibratory rollers.

Based on technical publications of Geodynamic (Sandström, Å. J. et al., 2004) and AMMANN; (Anderegg, R. et al., 2004) the difference in the approaches to the CCC are observed due to the totally independence in their work. IC technology developers estimate soil stiffness using a rational mathematical model of the ground-roller dynamic interaction. On the other hand, Geodynamic estimates a "cylinder deformation modulus" from the frequency domain evaluating the fundamental and first harmonic component of the roller-ground acceleration.

Recently, in the USA, Mooney, et al. (2005), contributed to the development of the CCC method. They explored other alternatives for analyzing the ground-roller system response for vibratory compaction from the time and frequency domain analyses, by attaching accelerometers to the roller as well.

All of the above mentioned ACT methods are focused on vibratory compaction, using instrumented drums. A different approach is required for static compaction which is also an extensively used technique for clayey soil compaction.

In the USA, an extension of CCC is being developed for static and vibratory compaction by Caterpillar, Inc. This approach is focused on the analysis of machine output energy as a function of change in physical soil properties. The assessment of soil compaction from change in equipment response is based on the fact that the mechanical energy to drive the roller is related to the physical properties of the material being compacted. Proponents of this approach proposed correlations between compaction energy and dry unit weight, water content, stiffness, and strength of the compacted soil. Caterpillar, Inc., Iowa Department of Transportation and Iowa Highway Research Board are working in the development, evaluation, and implementation of the technology for practice. Most of their research is focused on static compaction. This ACT is the only one available for static compaction.

In general, today the ACT includes: Global Positions Systems (GPS), machine sensors, microprocessors, reliable transduction systems, robust and inexpensive mass storage, and convenient interfaces for accessing and using acquired information. This technology utilizes systems that are attached to the compaction equipment that monitor the equipment performance and use the results to indicate to the operator whether or not the soil beneath the compactor has been compacted sufficiently to meet specifications. With this technology, compaction specifications are likely to transcend the conventional water content and density specifications now in widespread use to those required for the actual performance of pavements and embankments such as resilient modulus, stiffness, and shear strength (See Chapter 2).

Following are the general advantages and disadvantages of ACT:

Advantages

- QC/QA base on mechanical design parameters
- Compaction work is homogeneous
- Soft points in the subsoil are identified
- Over compaction is avoided
- Time is saved due to optimization of roller passes

- Reduction of premature concrete or asphalt-pavement rehabilitation
- Longer lifetime of the machine due to controlled vibrations
- Cost savings through reduction of resources used
- Recording of rolling operations.

Disadvantages

- It requires sophisticated and rugged equipment in a harsh environment.
- It requires some operator training.
- In the short term, the compaction cost increase.
- The estimated stiffness is not an independent parameter. The estimation of resilient modulus and strength requires calibration of the equipment using auxiliary lab and field tests.

At the moment, ACT is fairly popular in Europe, up the point that several countries have incorporated it in their construction specifications. In the USA, ACT is just being introduced. Briaud, et al. (2003) discuss the main factors that have lead to the delay in the implementation of ACT in the USA.

Considering the limitations of the current compaction QC/QA procedures and supporting the AASHTO, NCHTP and FHWA effort to implement the mechanistic-empirical design method in the USA (See Section 2.1), the FHWA produced a report titled "Accelerated intelligent compaction technology for embankment subgrade soils, aggregates base and asphalt pavement material" which is the basis for a pooled-fund research program to implement ACT in the USA consistent with the capabilities of the technology and the AASHTO-style construction QC/QA specifications). Additionally, Minnesota, Iowa, and Florida Departments of Transportation, in collaboration with equipment manufacturers, have devoted significant efforts on the evaluation of IC technology through the conduct of large-scale demonstration tests using the available technology.

First we review soil compaction mechanisms and the theoretical and practical bases of ACT. Then in the following sections, the above mentioned ACT methods are reviewed in detail, identifying their particular capabilities, advantages, and disadvantages.

4.3 *Soil compaction*

Compaction energy and the compaction method are parameters that determine the structure of compacted soils (See Section 2.4). In the field, the compaction energy per volume of soil is determined by the compaction equipment weight, compaction method, number of roller passes, and lift thickness. The compaction method is most often selected on the basis of the properties of the soil to be compacted and the requirements of the project. Sometimes, the compaction method is based on the equipment available.

4.3.1 Compaction methods

This section reviews conventional compaction methods. There are three compaction methods: Static, impact and vibratory compaction. Impact compaction method gives a greater force on the surface than the static and vibratory method. Impact compaction creates stress waves that travel from the surface into the soil generating higher stresses to larger depths 30 – 60 ft (10 - 20m) than static compaction. Vibratory tampers that are used in minor compaction jobs, also work with the impact principle (Forssbland, 1981).

Static smooth rollers, pneumatic-tire rollers, and static pad-foot rollers work with static pressures of different magnitudes on the surface. Pneumatic and pad-foot rollers combine pressure with a kneading effect. Pneumatic-tire rollers have a closing or sealing effect on the surface (Forssbland, 1981). However, these methods only allows for compacting to shallow depths and hence lift thicknesses must be controlled.

Static methods are appropriate for cohesive soils because of the low permeability of the clayey material that leads to abrupt increments of pore pressure when dynamic compressive stresses are applied, reducing the compaction effects because most of the compaction energy is used by remolding the material at constant volume.

Vibratory compactors work with a rapid succession of impacts against the surface of the ground (See Figure 4.1). Each impact produces stress waves which set the soil particles in motion. For particles near the compactor, the internal friction between the particles is virtually eliminated from the stress waves. Vibratory equipment combines vibration with static pressure that induces shear stresses in the ground. In general this method is more effective for granular soils. Recent development of this compaction method has led to the origin of different roller motions like combined oscillatory horizontal or torsional tractions and compression motion.

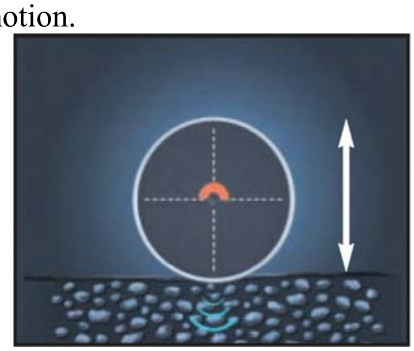


Figure 4.1 – Vibratory motion is caused by a rotating eccentric mass (Hamm brochure).

Horizontal oscillatory motion shakes the soil in the horizontal direction inducing shear deformations in the soil mass. Thus, horizontal oscillatory compaction is archived mainly by transmitted shear waves to the material. This type of motion is used for asphalt and cohesive soil compaction, and when compaction is conducted in the vicinity of sensitive structures because energy transmitted by vibration mode is much higher (Briaud et al., 2003). Additionally, horizontal oscillatory motion has a sealing effect on the surface (See Figure 4.2).

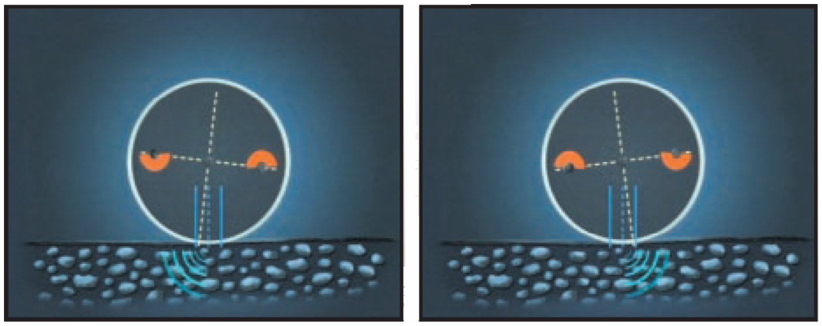


Figure 4.2 – Oscillatory motion that is associated with a torsional movement caused by two opposite rotating eccentric masses, which shafts are arranged eccentric to the axis of the drum (Hamm brochure).

On the other hand, combined dynamic motion uses both vertical vibratory and horizontal oscillation motion, having the capabilities of these two types of motions.

Vibratory and combined dynamic motions are more effective for sands and gravels, where there is a relatively low apparent cohesion. Thus, the state of motion during vibration makes quite effective vibratory compaction with a relatively low contact pressure ranging from 0.5 to 0.1 MPa (Forssbland, 1981). This allows using light vibratory compactors getting very high densities when the layers thickness is limited. For clays, heavy compactors are required to compact thick layers of soil (See Figure 4.3).

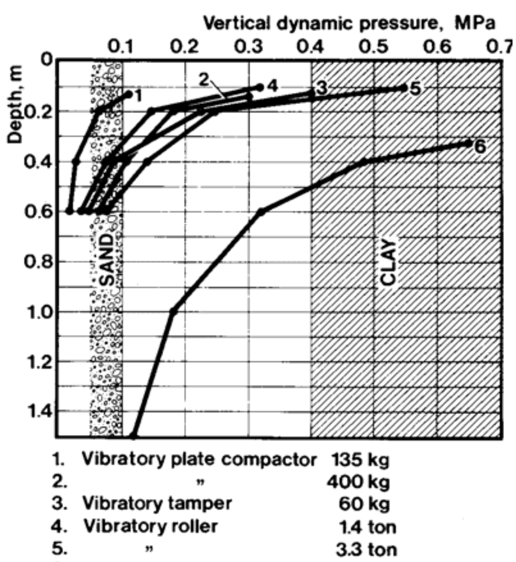


Figure 4.3 – Dynamic pressures at different depths obtained with different type and sizes of compactors (Forssbland, 1981).

Intelligent compaction rollers make use of the vertical vibration, horizontal oscillation, and torsional motion depending of the condition of the soil being compacted and in the compaction requirements. Selection of the motion type, roller speed, frequency, and amplitude of vibration are automatically or manually set. Motion frequency and roller velocity are tightly related with the requirement that two consecutive roller impacts have to be between 20 and 40 mm in order to have and homogenous spread of compaction energy (See Figure 4.4). The variation of amplitude is generated by equally splitting the eccentric mass in two pieces that rotate in the same direction. The maximum and minimum vibration amplitude is obtained when the angle between the two masses is 0° and 180° respectively. Intermediate amplitudes are obtained with angles between 0° and 180°.

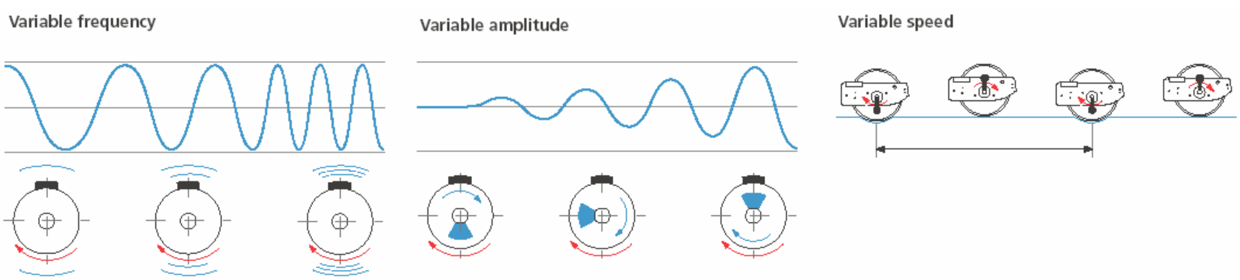


Figure 4.4 – Control parameters of Intelligent Compaction (AMMANN Brochure).

4.3.2 Soil compaction mechanisms

Olson (1963) provided a comprehensive qualitative explanation of the compaction process based on the effective stress theory of partially saturated soils.

Once the soil is spread and bladed, the material has a low shear strength because it is in a loose condition and the confinement effective pressure is low. The degree of saturation of the soil is low, which allows the existence of negative pore pressure (Olson, 1963; Fredlund, D.G et al., 1993).

When the roller passes for the first time, the shearing stresses developed between the particles reach the shearing strength of the contact surfaces, the contacts yield, the particles slides over each other, and the density of the soil increase. Simultaneously, both the total stresses and the pore pressure increase, but the pore pressure increases by less than the total stress because the soil is not saturated. Hence, the effective stresses between the particles increase as the roller passes. Large deformations of the soil occur underneath and in the vicinity of the roller until the increasing effective stresses give the soil sufficient shearing strength to resist the weigh of the roller and the vibratory force of the drum. Soil particles are rearranged and forced into a more dense packing. When the stress by the roller drum is released by the moving drum, the vertical total stress becomes zero. The soil expands slightly in the vertical direction with a simultaneous reduction in the lateral total stresses. Expansion of the soil is resisted somewhat by the development of negative pore-water pressures (capillary stresses). After a relative small expansion, the negative pore-water pressures and the residual lateral total stresses produce sufficient compressive effective stresses in the soil that the densified condition is maintained.

When the roller pass again, the compaction pressure is re-applied, shearing deformations again occur, there are increases in the total stresses and pore pressures, and the effective stresses develop sufficiently for the soil to resist the compaction pressure. The yielding of some of the inter-particle contacts during the stress application will result in higher densities that were developed during the first roller pass. Again, when the roller advances, the pressure is released, expansion of the soil is resisted by the negative pore-water pressure and by the total residual lateral stress.

The shearing strength of the soil increases as the roller passes due to the formation of increased number of particle contacts, locked-in lateral stresses, and increased capillary stresses. Eventually, the soil may become sufficiently strong so that relatively few inter-particle contacts yield with subsequent roller pass, except for small plastic deformations concentrated just underneath the drum. The soil has reached its maximum density for the particular compaction procedure. It is important to recognize that even during the last roller pass with the soil at a high degree of compaction, a distinct plastic deformation zone exists beneath the drum (Thurner, 2000). Hence, vibratory loading of the soil applied by the roller during compaction is likely to be much higher than loading of the soil during working conditions (See Section 2.3) and the unloading phase of the vibratory roller would be associated with the unload modulus and better simulate the modulus for working conditions.

For clean sands, compaction is effective by vibratory compaction with the sand at high water content (See Section 2.4.2). The vibration induces liquefaction. The combination of vibration and static pressure (weight of the roller) squeezes the water out and rearranges the particles into a more dense packing.

4.4 CCC using machine response

The Continuous Compaction Control (CCC) method for static compaction using machine response is being developed by Caterpillar and represents the only ACT available for QC/QA of static compaction.

4.4.1 Current INDOT Specifications

The Indiana Department of Transportation (INDOT) has as part of its construction specification the use of proof-rolling for compaction QUALITY CONTROL purpose (See Section 200 Earthwork of 2006 Standard Specifications):

> Item 203.26 Proofrolling “The test consists of a pneumatic tire roller or a loaded tri-axle dump truck passing over the compacted area to detect under-compacted zones. All roller marks, irregularities or failures are corrected afterwards.

The specification used to, but no longer, recommends "subjectively evaluating the engine sound, considering that for a well compacted surface, the engine has a consistent, unlaboring sound. Conversely, on soft areas the engine will have to labor in order to pull the roller or truck through the spongy area." (See Figure 4.5)

Although the proof-rolling test reduces the uncertainty of the compaction quality, this test does not support the compaction work with a real-time feed back due to this test is just performed at the end of the compaction, when the compaction equipment has moved to another location. Furthermore, the non-systematized method and the uncoupled construction procedure make the method time consuming and unreliable to control compaction quality.

Figure 4.5 – Proof-rolling field test (INDOT Standard Specifications)

4.4.2 Principles for CCC

A more robust technology that is still under development has been introduced by Caterpillar, which is based on the premise that changes in equipment response are related to the physical properties of material being compacted. For this purpose, monitoring sensors are attached to the engine of the compaction machine, a differential local positioning satellite system allows for clearly mapping the work area and identifying the degree of compaction over the entire work site. The computational process is supported by an on-board digital system (See Figure 4.6).

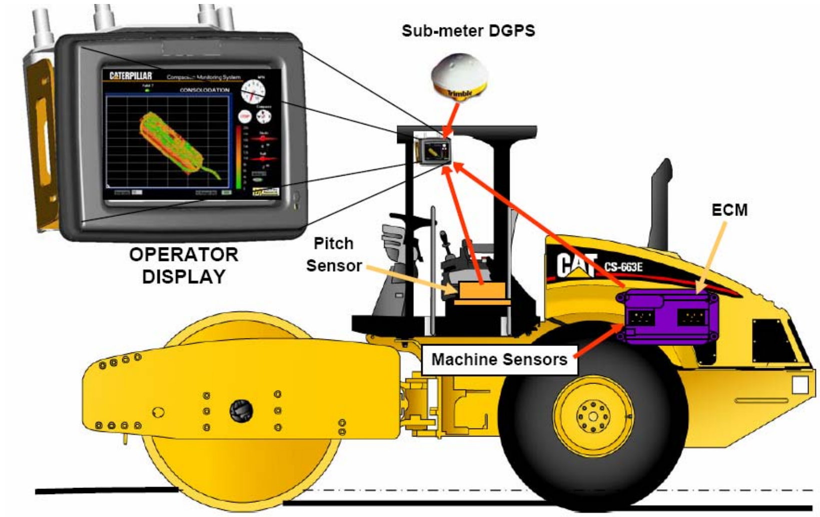


Figure 4.6 – Static Intelligent compaction approach (White et al., 2005)

Use of high level technology makes it possible to have the compaction machine also serve as a measuring device, providing real-time feedback during compaction.

4.4.3 Roller-ground interaction

The study of the wheel-ground interaction started about hundred fifty five years ago, with the work of Grandvoinet, Gerstner, Coulomb, Morin and Reynolds (Bekker, 1956). Later work in the fifties and sixties was based on soil behavior modeling using empirical coefficients without any physical meaning and concern about the analysis of the stress-strain behavior of soils under the action of a wheel (Bekker, 1956, Ageikin, 1987, White et al., 2004, White et al., 2005).

Four different cases of wheel-ground interaction are typically considered. They involve the application of: 1) a rigid wheel to a rigid surface; 2) a rigid wheel to and elastic or plastic surface; 3) an elastic wheel to a rigid surface; and 4) an elastic wheel to an elastic or plastic surface (Ageikin, 1987). The case associated with the compaction roller and soil being compacted is represented by the second case where the complexity of the interaction due to flexibility of the wheel is avoided.

Under a static compaction, Schuring (1966) defined the energy loss in a two-dimensional configuration by the following expression (See Figure 4.7):

$$E_s = \frac{1}{L-i}\left[R(i-L)+\frac{M}{r}\right] = \frac{1}{l-i}rb\left[i\int_{\theta_1}^{\theta_2}\sigma_h d\theta + \int_{\theta_1}^{\theta_2}\sigma_v \theta d\theta\right] \qquad (4.1)$$

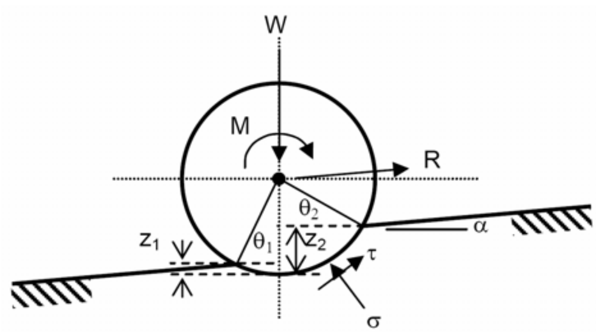


Figure 4.7 - Simplified two-dimensional free body diagram of stress action on a drum (White et al., 2005).

where, R is the drawbar pulling force, M is the torque applied to the roller, r is the roller radius, L is the horizontal distance traveled by the roller, i is the wheel slippage, b is the roller width, σ_h and σ_v are the horizontal and vertical stress, respectively acting on the roller, and θ_1 and θ_2 are the interface contact angle, which is function of the sinkage. The expressions for θ are given by

$$\theta i_i = \cos^{-1}\left(\frac{r - zi}{r}\right) \qquad z_i = \left[\frac{3W}{(3-n)(k_c + bk_\phi)\sqrt{D}}\right]^{\frac{2}{2n+1}} \qquad (4.2)$$

where, D is the roller diameter, W is the roller weight, b is the roller width, and k_c, k_f, n are deformation parameters. These last three parameters are difficult to determine, requiring plate load tests with multiple sized plates and extrapolation, which complicates the calculation of Energy loss. Furthermore, the inherent variability of soils and the error associated to the imperfect modeling of the ground-roller interaction may even make it impossible to estimate the accurate parameters (Bekker, M.G., 1969). Thus, an alternative way to evaluate energy loss is the empirical approach, looking toward the determination of relationships among machine response and soil properties (White et al., 2004).

The gross power (P_g – energy/time) required to move the compaction roller through the uncompacted layer of fill can be given by the following expression.

$$P_g = P_{ml} + P_s + P_{sa} \qquad (4.3)$$

Where P_s represents the portion of the power need to overcome resistance from moving the compactor through the soil, P_{sa} is the additional machine power only associated with sloping grade, and P_{ml} is the internal machine power loss. From the previous expression, energy loss is given by

$$E_s = P_g t - P_{ml} t - WV\left(\sin\alpha + \frac{a}{g}\right)t \qquad (4.4)$$

Where E_s is the compaction energy, a is the acceleration, g is the acceleration of gravity, α is the slope angle, t is time and V is velocity (White et al., 2004). The term E_s includes slippage.

There are two models used to explain the limiting wheel-soil traction for the particular case of sheep pad rollers or pneumatic compaction equipment (See Figure 4.8a). The first model states that the traction reaches its limit (slipping) due to the shear of the soil trapped between the projections of the tread and due to sliding of the projection of the tread on the soil. The second model considers that the wheel-soil traction attains the limiting condition as a result of the soil losing its bearing capacity when the total loading on the wheel becomes equal to the bearing capacity of soil. In this case, the shear force of the soil during wheel slip occurs at a certain distance from the contact surface (See Figure 4.8b). The actual value of the wheel-soil traction limit is equal to the lower of those obtained by the two models (Ageikin, 1987). For the compaction case the traction limit is usually given by the second model that is presented at the beginning of the compaction of a loose lift.

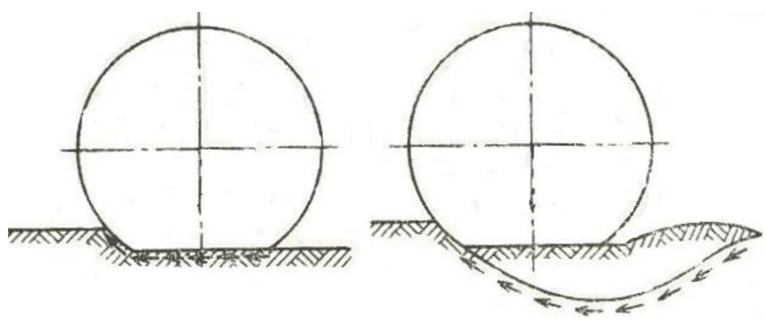


Figure 4.8 - Models for wheel-soil limiting traction (Ageikin, 1987)

Empirical approach

Iowa State University, sponsored by Iowa Department of Transportation, Iowa Research Board, and Caterpillar with collaboration of FHWA is working on the evaluation and implementation of the Caterpillar monitoring-machine-response technology. The main findings of their research presented in the draft version of the report “Field Evaluation of Compaction Monitoring Technology: Phase I” (White, D. et al., 2004). Most of their work is focused on the application of the technology to static compaction.

As discussed in the previous section, the method is based on empirical relationships among power (gross power, P_g and net power, P_s) and soil properties that are relevant in the earth-structure behavior, like modulus and strength. These relationships are controlled by soil type, water content, compaction energy, and compaction method (See Section 2.4).

To evaluate the technology, these relationships were first explored in the laboratory, where controlled conditions allow for evaluating the effects of the water content, density, and compaction energy on a given soil. Once the effects of these factors are determined, large scale tests using monitored machine response will be reviewed.

Considering that CCC using machine response has been applied mainly for static compaction, the following discussion will be focused on clayey soils.

Laboratory work

Chapter 2 explored the effects of soil type, water content and compaction energy on the resilient modulus (See section 2.4). Particularly, for clayey soils it was pointed out that resilient modulus decreases as water content increase. Additionally, clayey soils compacted with high compaction energy exhibit a high resilient modulus on the dry side of the optimum water content, while the trend reverses at water contents wet of the optimum. The effect of increased compaction energy is to decrease optimum water content and increase the maximum dry density.

In order to explore the relationships among compaction energy, strength, and stiffness, White, D. et al. (2004) performed compaction tests with several compaction energies. After compaction, the specimens were tested for their unconfined compression strength. The common observation in the linear regression analysis was that increasing the compaction water content considerably reduced the scatter of the data, which reflects the important role of the water in the soil behavior. The results of the regression analysis for a low plastic clay (CL) soil with a liquid limit of 29% and plasticity index of 13% are summarized by the equations 4.5 and 4.6, with R^2 values for the strength, S_u, and modulus, E_{50}, are 0.74 and 0.54, respectively.

$$S_u = 326 Log(E) - 15.5(w\%) - 577 \tag{4.5}$$

$$E_{50} = 41262 Log(E) - 8062(w\%) - 2362 \tag{4.6}$$

where: E_{50} is the secant Young's modulus for axial stress equal to 50% of the strength, E is the compaction energy in kJ/m^3 and w is the water content in percent. The higher scatter of the modulus-energy and strength-energy linear regression analysis is because both dry side and wet side specimens were included in the data analysis. Discussion in Chapter 2 indicated that dry and wet side compacted soils presented a totally different resilient modulus-energy and strength-energy trends (See Section 2.4.1).

Field large scale tests

Working with the same soil used in the laboratory work, White, D. et al. (2004) conducted large scale test on seven 20m long strips at a Caterpillar facility site. The main purpose of the research was to evaluate the Caterpillar machine-monitoring technology. The construction operations for the site involved:

- Aerate/mix the soil.
- Adjust moisture conditions in the soil with a water truck to obtain water content ranging from 8% to 17%. The water content in each strip was roughly the same.
- Remix soil after adding the water.
- Spread and blade to level surface with lift thickness ranging between 300 and 650 mm. The lift thickness of each strip was roughly the same.
- Compact the soil with 6 to 10 passes monitoring the gross power along each pass. In this particular field test, an 11-ton-padfoot roller was used.
- Determinate dry density using drive tube (reference value), nuclear gauge, and sand cone; water content using oven dry method; strength using dynamic cone penetration test; and stiffness using GeoGauge.

As in the case of the laboratory work, multiple regression analyses were conducted between the power output (gross and net power) from the compaction monitoring system and the soil properties from field measurements. The main steps of the regression analysis were:

- Enter data into ArcGIS, a Geographic Information System software package.
- Assemble the power data by the number of passes for each test strip using ArcMap.
- Determine engine power values for each test strip.
- Match power values at points that correspond to test points.
- Plot paired scatter plots to observed trends and propose regression models
- Perform simple and multiple regression analyses.

Figure 4.9 presents machine gross power values as a function of roller passes.

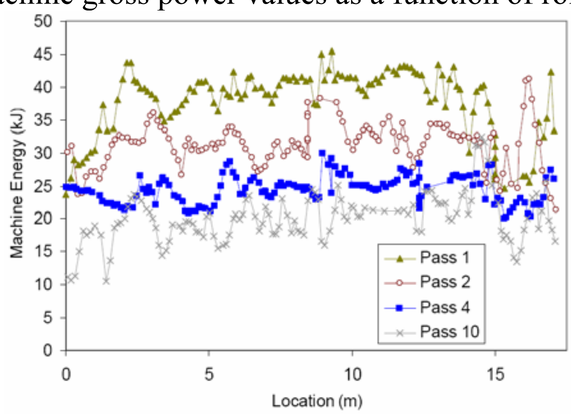


Figure 4.9 – Machine gross power values as a function of roller pass (White, D. et al., 2005).

Based on correlations with soil properties, it is possible to estimate the degree of compaction. Considering that strength and resilient modulus are the parameters that determine the mechanical behavior of a compacted earth-structure, these two should be the criteria to determine the accuracy of the equivalence between machine energy and compaction degree. Figure 4.10 presents the output results of compaction monitoring, showing the evolution of the compaction process.

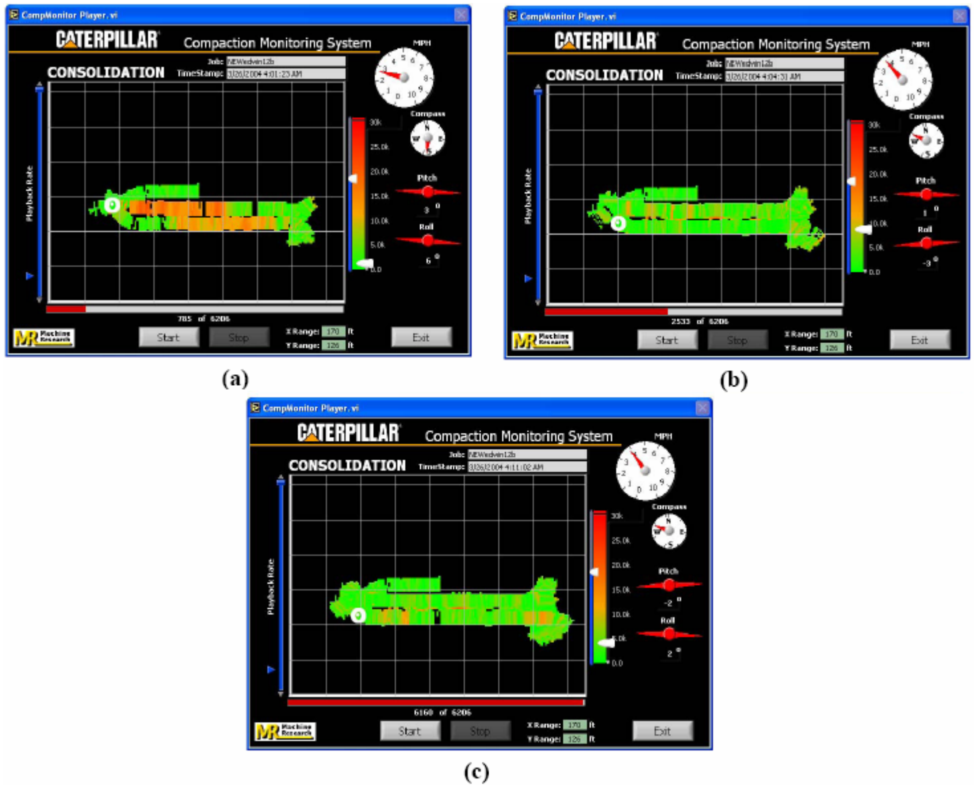


Figure 4.10 – Monitoring output machine energy after a) 1, b) 4 and c) 10 pass on Edwards Test Facility (White, D. et al., 2004). Red color is associated with low degree of compaction and the green color is associated with high degree of compaction.

4.4.4 Modulus evaluation of the current state of machine response CCC technology

Obtaining detailed information of state-of-the-art intelligent compaction is difficult because this information is considered confidential by the companies involved in the research and development of advanced compaction technology.

The evaluation of CCC based on machine response monitoring was obtained from a draft version report titled “Field Evaluation of Compaction Monitoring Technology: Phase I” (White, D. et al., 2004) and the TRB paper titled “Real-time compaction monitoring in cohesive soils from machine response” (White, D. et al., 2005).

The evaluation of the CCC machine monitoring was done on three pilot tests. Two of the sites were located in Peoria, Illinois, at Caterpillar facilities. The third project was an actual earthwork grading project in West Des Moines, Iowa.

The Caterpillar technology is based on a net power value and number of roller passes as an indicator of compaction. The main limitations identified by White, D. et al., 2004 are:

- In some cases there was an erratic variation of machine energy. At some points the machine response was greater than that obtained in previous roller passes (See Figure 4.11). This observation was reported in two of the three field tests. This phenomenon may be related with soil variability or internal machine loss. White, D. et al., 2004 recommended additional research to address this problem..

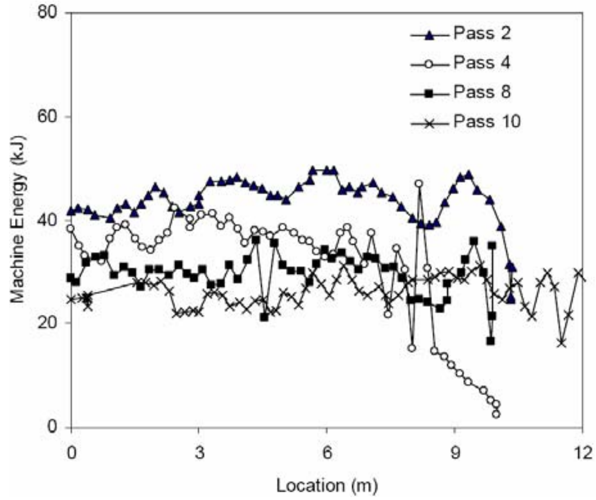


Figure 4.11 – Erratic variation of machine response.

- The net power given by the Caterpillar system, calculated from the gross power, is not sensitive to the loss associated with sloping grade. White, D. et al., 2004 recommended refining the estimation of net power to correctly reflect the mechanical properties of the soil being compacted.

- The correlation between soil properties and net power is weak (i.e. regression analysis with low R^2). This problem is associated with the previous problem relating to the estimation of net power.

- In work areas with high variability of soil properties, the GPS mapping of the point where the compactor interacts with the ground does not exactly correspond to the point.

4.4.5 Final comments

CCC based on machine response monitoring is a systematized QC/QA method that would be a more appropriate tool than the proof-rolling test. However, the current state of this technology requires refinement before being implemented in practice. Based on the information available from Iowa State University research (White, D. et al., 2004), further work is needed on: the review and analysis of the estimation of net power from the gross power; statistical analysis of the power out data; GPS mapping of the ground area being compacted; and establishing criteria for zonation of degree of compaction.

The main advantages of the method with compared to the proof-rolling test are:

- It is a method totally coupled to the construction process, which allows having 100% coverage with information provided in real time.
- It is a totally systematized method independent of personal judgment.

After refinement it is expected that the ability of the technology would be constrained to provide a relative degree of compaction. Estimation of strength and modulus, parameters that control the earth-structure performance, could only be estimated based on preexisting correlations or on correlations obtained for the particular site. This aspect could represent the main disadvantage of the technology, considering that the microstructure of the soil, reflected in the strength and stiffness, is function of soil type, water content, compaction energy, and, particularly in clays, compaction method (See Section 2.4, Seed, H.B., 1959; Bell, J.R., 1977).

The formulation of soil property–power relationships could be quite complex and time consuming, and they may be limited to particular conditions considering:

- The technology is totally based on empirical energy-soil properties relationships that are difficult to be generalized for cohesive soils and non-plastic silty sands because their structure is sensitive to water content, compaction method (i.e. static, kneading, impact, vibration) and compaction procedure (i.e. roller velocity) (See Section 2.4; Seed, H.B, et al., 1959; Olson, R.E., 1963; Bell, J.R., 1977).

- Machine energy losses may not be constant with loads applied to the engine and drive train due to friction.

- Change of environmental conditions, especially temperature, affects the viscosity of equipment lubricants, which would be reflected in the machine performance affecting the calibration equations.

Thus, the main disadvantage of this method seems to be the reliability of the empirical equations. Considering this fact, further research may need to focus on estimation of modulus and strength (See Section 4.3.4). It may be more practical to have just a relative measure of compaction degree, which still would improve de overall compaction and construction process proving a better QC/QA than the existing one.

4.5 CCC for vibratory compaction

CCC technology for vibratory compaction was developed in Europe at the end of the seventies. This method conducts QC/QA based on the analysis of acceleration records obtained by the instrumentation of the roller with accelerometers. Subsequent research by compaction equipment manufacturers and research centers in Europe and in the USA has been focused on the improvement of the technology. These research works have been conducted in an independent and confidential manner, given origin to different approaches.

4.5.1 Principles

The main principles of CCC methods for vibratory compaction were introduced in Section 4.2. European CCC technology estimates deformability parameters of the soil being compacted considering the drum as a loading instrument (See Figure 4.12), similar to the loading plate of the FWD test (See Section 3.2). Thus, accelerometers are attached to the drum and frame of the roller. Having acceleration record during compaction along with the pertinent characteristics of the roller (i.e. weight of the equipment, operation frequency and drum dimensions), deformation parameters are estimated and correlated with resilient moduli obtain from FWD.

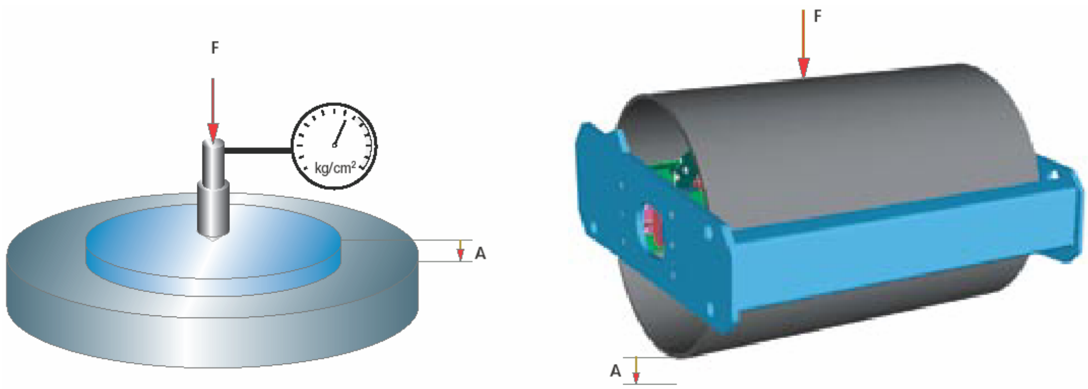


Figure 4.12 - Estimation of deformability parameters of the soil being compacted using the compaction equipment (AMMANN brochure).

Based on the technical papers from European equipment manufacturer representatives (Sandström, Å. J. et al., 2004; Anderegg, R. et al., 2004), two different methods to calculate deformability parameters from the acceleration records were identified. The first method calculates soil stiffness (k_s) based on the mathematical analysis of the roller-ground interaction and clearly identifies the main forces that govern equilibrium of the system. The second approach estimates a relative parameter called “cylinder deformation modulus E_c” from the frequency domain, evaluating the fundamental and first harmonic component of the roller-ground acceleration. These parameters (i.e. k_s and E_c), then can be related to the resilient modulus. This last step is the most critical part as we will discuss in the following sections.

Another alternative presented by Mooney et al. (2005) is based on the roller-ground interaction as captured by the acceleration record.

4.5.2 Roller-ground interaction

This section introduces the concept of dynamic roller-ground interaction. It is the basis for estimating deformability parameters of the soil from the roller-ground response. The roller-ground interaction is excited by unbalanced masses (m_e) located in the roller drum. In the mathematical modeling of the roller, the equipment is composed by the roller frame mass (m_f) and the roller drum mass (m_d). The drum is supported by suspension elements that have stiffness (k_t) and damping properties (c_t) (See Figure 4.13). The mechanical connection between the drum and the frame, causes the frame mass (m_f) to behave as a lowpass filter (Anderegg, R. et al., 2004). Figure 4.13 presents the mathematical model and equations 4.7 and 4.8 formulate the force-equilibrium condition of the roller-ground system assuming that the stresses applied to the ground are in the elastic range (linear elastic soil behavior).

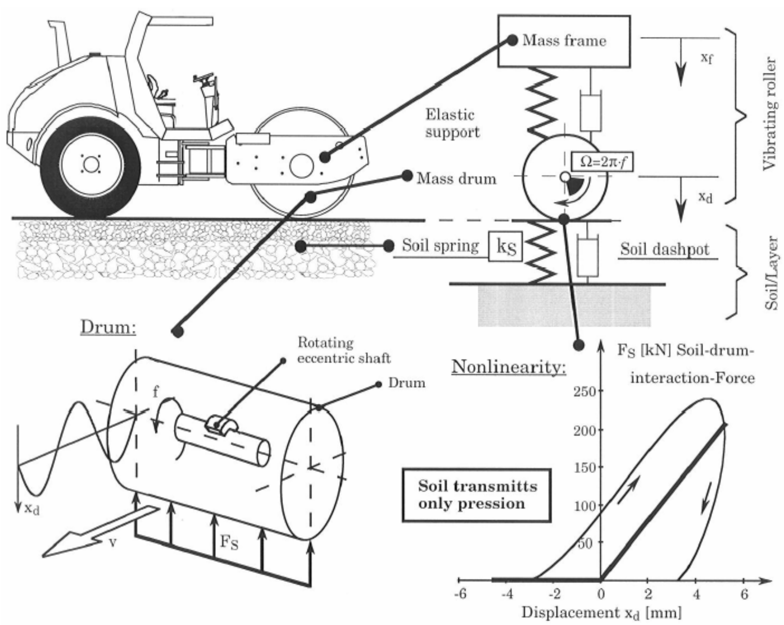


Figure 4.13 – Roller-ground system (Anderegg, R. et al., 2004)

$$\begin{aligned} m_d \ddot{x}_d + Fs &= m_e r_e \Omega^2 \cos(\Omega t) + k_t\left(x_d - x_f\right) + c_t\left(\dot{x}_d - \dot{x}_f\right) + m_d g \\ F_s &= m_e r_e \Omega^2 \cos(\Omega t) + k_t\left(x_d - x_f\right) + c_t\left(\dot{x}_d - \dot{x}_f\right) + m_d g - m_d \ddot{x}_d \end{aligned} \qquad (4.7)$$

$$F_s = k_s x_d + c_s \dot{x}_d \quad \text{, if } F_s > 0 \qquad (4.8)$$

where:

m_d is the drum mass
F_s soil reaction
$m_e r_e$ is the eccentric moment of unbalanced mass
x_d is the drum vertical displacement
k_t is the suspension stiffness
c_s is the soil damping

f is the excitation frequency
Ω is the circular freq. = $2\pi f$
x_f is the vertical frame displacement
k_s is the soil stiffness
c_t is the suspension damping.

The effect of the frame mass is considered only by the spring forces generated by the relative displacement between the frame and drum; no damping between the frame and drum is considered. Simplifying the model, Equation 4.9 presents the equilibrium equation of the roller-ground system neglecting the dynamic forces of the frame suspension and considering only the static frame mass.

$$\begin{aligned} m_d \ddot{x}_d + F_s &= m_e r_e \Omega^2 \cos(\Omega t) + \left(m_f + m_d\right) g \\ F_s &= m_e r_e \Omega^2 \cos(\Omega t) + \left(m_f + m_d\right) g - m_d \ddot{x}_d \end{aligned} \tag{4.9}$$

where: m_f is the roller fame mass.

The resulting vertical displacement of the ground (x_d) will be sinusoidal as the excitation roller force, with the same frequency of the excitation but with a delay in the soil response represented by a phase lag (ϕ) (Heukelom, 1961). The maximum soil reaction, $F_{s\ max}$ can be calculated using Equation 4.9. Figure 4.14 presents the variation of $F_{s\ max}$, vibration amplitude (A) and phase (ϕ) with excitation frequency (f).

Figure 4.14 shows that as the amplitude of the excitation increases (increasing $m_e r_e$), the response of the roller-ground system becomes highly nonlinear. This nonlinear behavior is mainly due to the loss of contact between the roller and the ground. Another important aspect to notice is that the response resonant frequency decreases with the excitation level. Therefore, the roller-ground motion can be classified in the following types (See Figure 4.15):

- Linear behavior: This state occurs when the drum is always in contact with the ground. The Fourier Spectra of the soil reaction will show that the energy is concentrated in one frequency component. This condition is identified as load operation.

- Periodic nonlinear loss of contact: This condition occurs when the drum periodically lifts off the ground. Then there are time intervals where the soil reaction is zero. The Fourier Spectra of the soil reaction will show the spreading of the energy in harmonic frequency components (multiples of the fundamental frequency component, i.e. 1, 2, 3, 4)

- Bouncing/rocking: In this state the machine shows signs of jumping, where the eccentric mass can have a complete cycle while the drum is still the air. In this operation condition the space between impact points increase and the soil may be loosened. The machine chassis is highly affected by the chaotic vibrations. The Fourier Spectra of the soil reaction will show the spreading of the energy in subharmonic frequency components (fraction of the fundamental frequency component, i.e. 1/2, 1, 3/2).

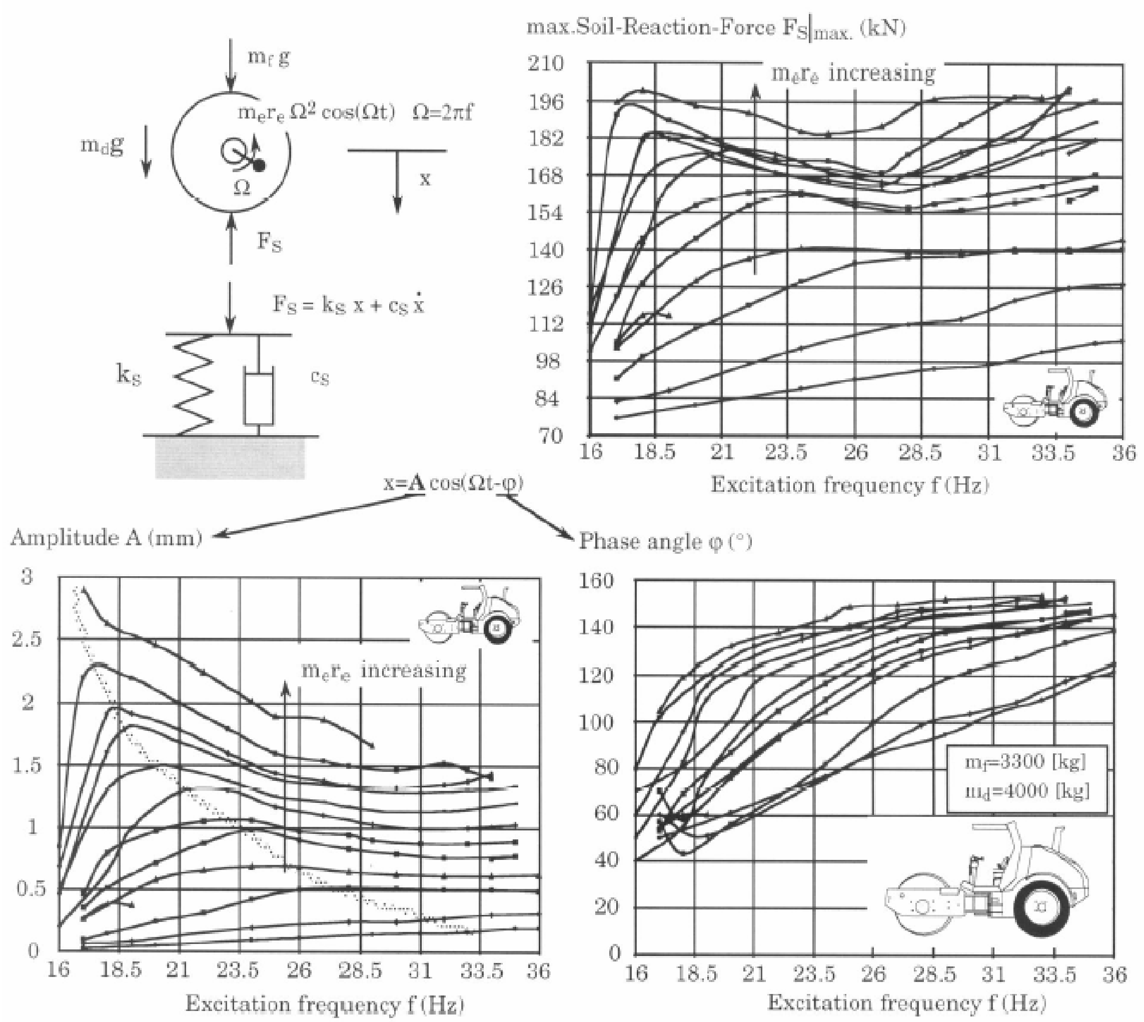


Figure 4.14 – Simplified model of the roller-ground system neglecting dynamic forces of the frame suspension for a roller-ground model with linear elastic soil behavior (Anderegg, R. et al., 2004).

The transition from one state of motion to another, keeping constant the excitation level, can be caused by the hardening of the soil during compaction (See Section 4.3.2). Also, it is possible to estimate that the drum is in permanent contact with the ground if the soil reaction is less or equal to two times the static weight (See Equations 4.10 and 4.11).
Permanent contact

$$F_s|_{\max} \leq 2(m_f + m_d)g \tag{4.10}$$

Periodic loss of contact

$$F_s|_{\max} > 2(m_f + m_d)g \tag{4.11}$$

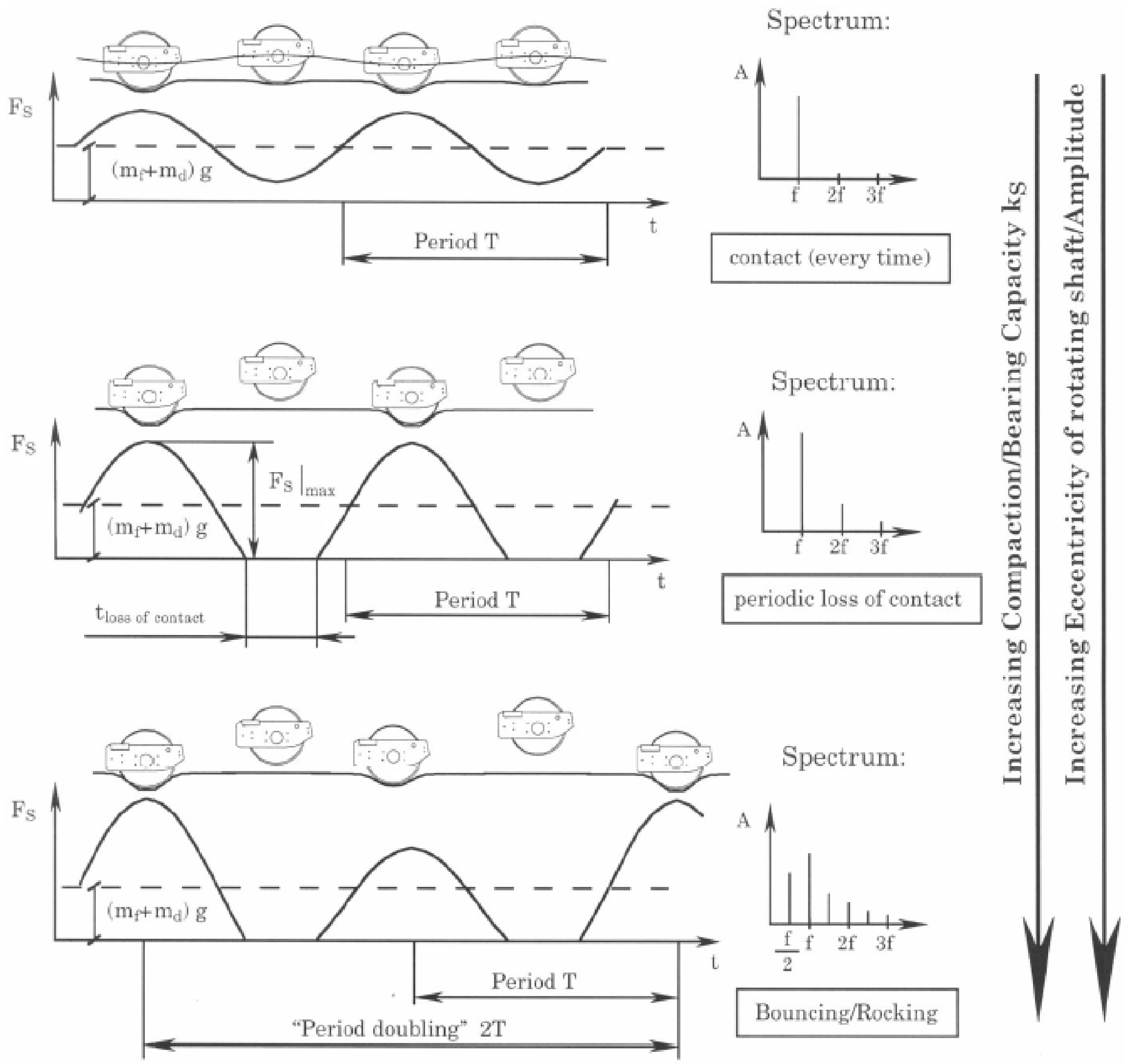


Figure 4.15 – Drum state of motion (Anderegg, R. et al., 2004).

The purpose of the Intelligent Compaction is to automatically control the compaction equipment, based on the measured vibration, to achieve optimal entry of compaction energy into the ground. This energy level changes as the soil is compacted because the soil stiffness is increasing (See Figure 4.16; See Section 4.3.2).

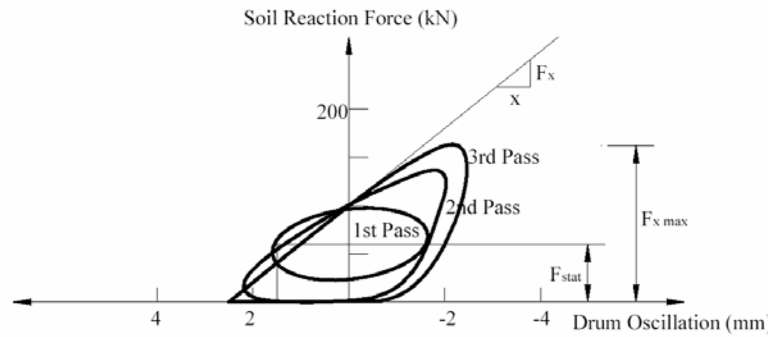


Figure 4.16 – Hardening of soil through the compaction process. The compaction energy of each roller pass is given by the area enclosed by the loops (Briaud et al., 2003)

For example, the compaction of a loose subgrade requires starting with high amplitudes and low frequencies in order to have compression waves with high amplitude (high compression energy) and long wave length to get deeper. As the compaction process continues the soil becomes stiffer and the applied vibration amplitude should be decreased while the vibration frequency is increased to concentrate compaction in a shallow depth. All of these changes of motions are automatically controlled by Intelligent Compaction equipment (See Figure 4.17).

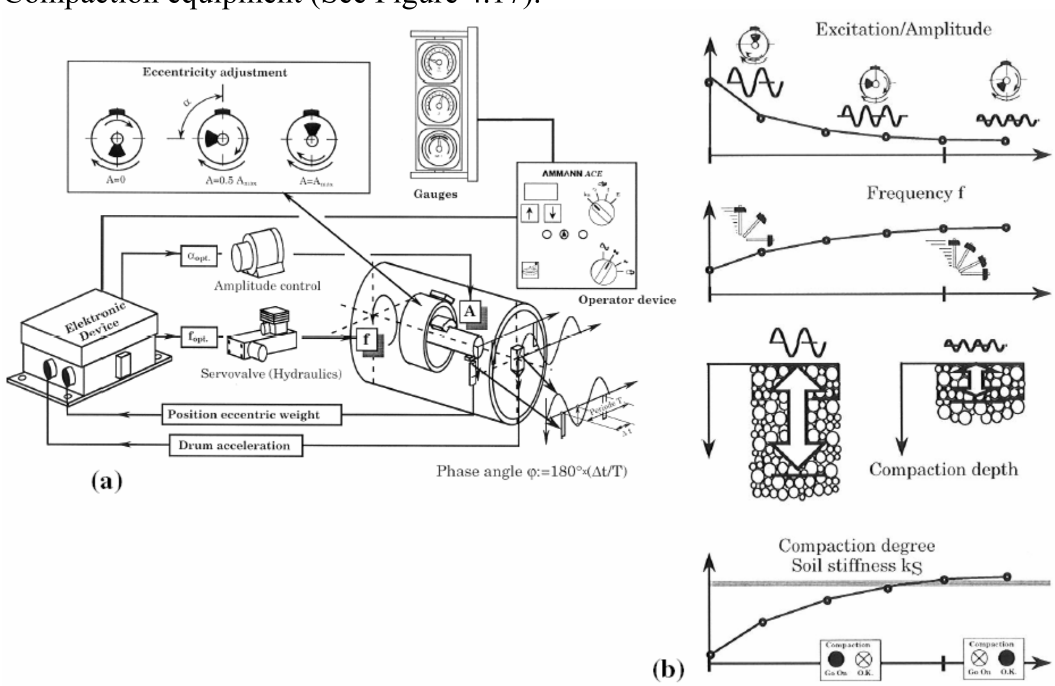


Figure 4.17 – Intelligent compaction system sketch. a) ACE: AMMANN compaction equipment. b) Automatic adjustment of amplitude and frequency while increasing compaction (Anderegg, R. et al., 2004).

4.5.3 Soil stiffness estimation from roller-ground response analysis

Pioneer work of Heukelom (1961) on ground stiffness associated to impact type loading by traffic showed that the soil reaction force is mainly given by the stiffness and to a much lesser degree in damping. Sustained vibration condition introduces a much greater damping effect to the soil reaction.

Based on the roller-ground interaction model (See section 4.5.2), the stiffness of the soil is calculated considering the net force acting on the roller-ground system when the drum velocity is zero (this allows for neglecting soil damping), and on the vibration amplitude.

Equations 4.12 and 4.13 lead to the estimation of soil stiffness (k_s) for vibration with periodic loss of contact (no bouncing or rocking) and vibration without loss of contact, respectively (See equations 4.10 and 4.11).

$$k_s = \frac{F_s\big|_{\dot{x}=0} - \left(m_f + m_d\right)g}{A} \tag{4.12}$$

$$\Omega = 2\pi f$$

$$F_s = \Omega^2 A m_d + m_e r_e \Omega^2 \cos(\phi) \tag{4.13}$$

$$k_s = 4\pi^2 f^2 \left(m_d + \frac{m_e r_e \cos(\phi)}{A} \right)$$

where A is the maximum roller amplitude in mm.

The stiffness obtained from a vibration mode associated with a periodic loss of contact is frequency independent. This means that the QC/QA should be done with this type of motion. Anderegg, R. et al. (2004) report experimental data that support the independence of k_s presented by Equation 4.12 (See Figure 4.18). It was observed that only at high frequencies the system started to become dependent on the excitation frequency. The test was done with a 20 ton (95 kN) tandem roller compacting a homogeneous subgrade.

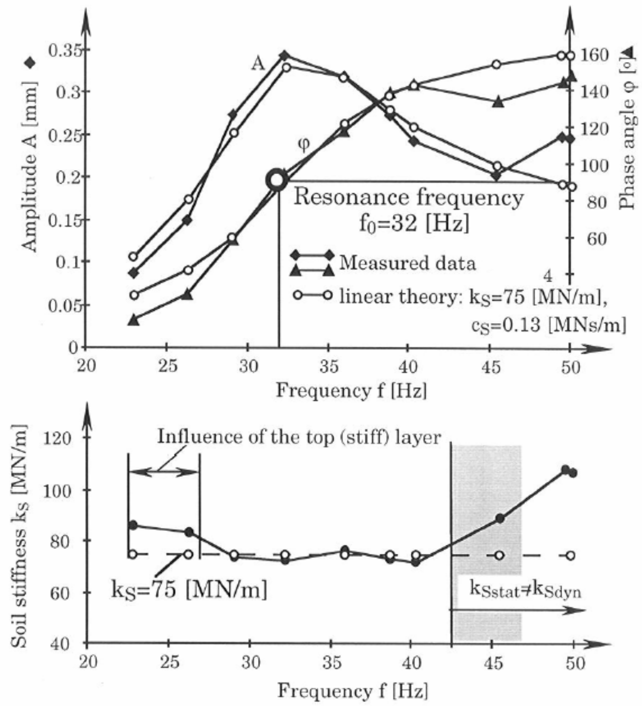


Figure 4.18 – Evaluation of the mathematical of roller-ground system and evaluation of the k_s - f relationship (Anderegg, R. et al., 2004). The higher stiffness of the top layer induces a dispersive wave propagation behavior (See Section 3.3.1).

The stiffness is a loading-area-size-dependent parameter. The relationship between k_s, associated with a cylinder on a elastic half space, and elastic modulus E was determinated by Hertz and Lundberg with the following equation (Anderegg et al., 2004):

$$k_s = \frac{EL\pi}{2\left(1-\upsilon^2\right)\left[2.14+\frac{1}{2}Ln\left(\frac{\pi L^3 E}{16\left(1-\upsilon^2\right)\left(m_f+m_d\right)Rg}\right)\right]}\left[MN/m\right] \qquad (4.14)$$

where:

E is the Young modulus [MN/m^2]

L is the drum width [m]

R is the drum radius [m]

ν is the Poisson ratio [-]

g is the gravity acceleration [9.81 m/s^2]

The previous equation allows for estimating the modulus, E, having the stiffness, k_s, and the Poisson's ratio, ν, of the soil being compacted and the properties of the compaction equipment.

Another alternative to estimate soil modulus is to establish an empirical relation between the stiffness and the soil deformability parameters obtained from auxiliary field test. In Chapter 3 explored some field tests that allow estimating soil elastic modulus like the

Falling Weight Deflectometer FWD. AMMANN reported a k_s-M_E relationship using plate load test from Federal Institute of Technology in Zurich (M_E is plate load modulus) (See Figure 4.19).

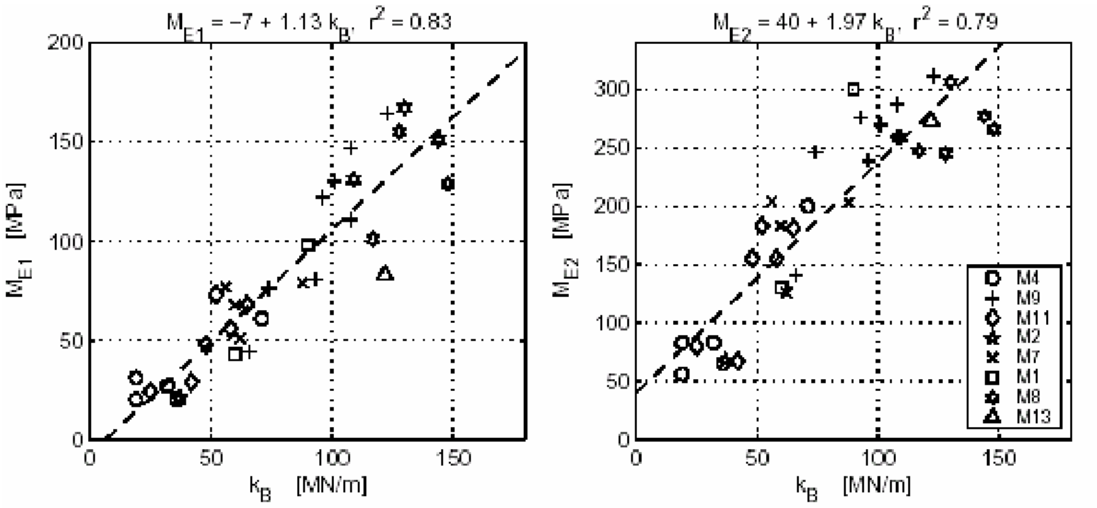


Figure 4.19 – Empirical relationship between soil stiffness from roller and soil elastic modulus (Anderegg, R. et al., 2004).

4.5.4 Deformability parameters estimation from time and frequency domain analysis of acceleration records

Estimation of relative deformability parameters based on time and frequency domain are a simpler approach to the roller-ground interaction analysis.

Geodynamik estimates a deformability parameter called "cylinder deformation modulus E_c considering the drum as a dynamic load test (i.e. similar to the FWD test). Thus, stiffness is calculated as the ratio of force (F) to displacement (s). However, the method considers that the impact force is proportional to the first harmonic component of the vertical acceleration of the drum, and the displacement during the drum impact is equal to the double integral of the fundamental acceleration component (Sandström et al., 2004). This double integration is actually never conducted, and the concept of pseudo-acceleration is applied (i.e. determination of acceleration from an equilibrium equation that neglect damping of the system) (See Equation 4.15)

$$E_c = c_1 \frac{F}{s} = c_2 \frac{A_1}{\left(A_o / \Omega^2\right)} \tag{4.15}$$

where:

Ao is the amplitude of the fundamental frequency component of the vibration
A_1 is the amplitude of the first harmonic frequency component of the vibration
c_1 and c_2 are constants

Based on Equation 4.15, Geodynamik defined the Compaction Meter Value (CMV). CMV is the parameter used to conduct CCC (See Equation 4.16).

$$CMV = 300\frac{A_1}{A_o} \tag{4.16}$$

The CMV parameter is frequency dependent, and additionally varies from roller to roller. Thus, the estimation of resilient modulus from correlation with field test requires obtaining CMV keeping the excitation frequency constant. However a standardized roller operated at a standardized frequency setting could be used, like the portancemeter that is a narrow vibrating standardized roller used in France for field measurement (Briaud, 2003).

Another simple analysis method of the roller-ground system is presented by Mooney et al. (2005). The approach is based on the consideration that the roller-ground interaction is captured by the roller drum acceleration record. Thus, the properties of the soil being compacted can be obtained through the time and frequency domain analyses of the acceleration record during compaction.

Mooney et al. (2005) conducted compaction field tests attaching accelerometers to the drum and frame. In the first phase test four homogenous materials were compacted; rubber, clay, asphalt and concrete. In the second phase, large scale compaction test were conducted in the field using sand and crush rock. Conventional field tests were carried out to support the experimental work.

The data collected by Mooney et al. (2005) compared peak acceleration obtained during the compaction of the different materials. However, in the frequency range of the roller operation (around 25Hz) a clear trend was not identified and the collected data appears to be a function of the soil type. However, considering that the depth of the bedrock is an important parameter in the characterization of roller-ground response, Mooney pointed out that bed rock position is an important factor to evaluate the peak accelerations

Additionally, Mooney et al. identified that the Total Harmonic Distortion (THD) in the frequency domain is a highly sensitive parameter in the evaluation of the compaction state of a soil (See Equation 4.17 and Figure 4.20). However, this method of analysis still needs further development. The effects of the soil profile and bedrock position also strongly affect the response of the roller-ground system. Thus, it is important to incorporate them at the start of the analysis process.

$$THD = \frac{\sqrt{A(f_2)^2 + A(f_3)^2 + \dots\dots + A(f_N)^2}}{A(f_1)} \tag{4.17}$$

where $A(f_N)$ is the acceleration amplitude of the N frequency components

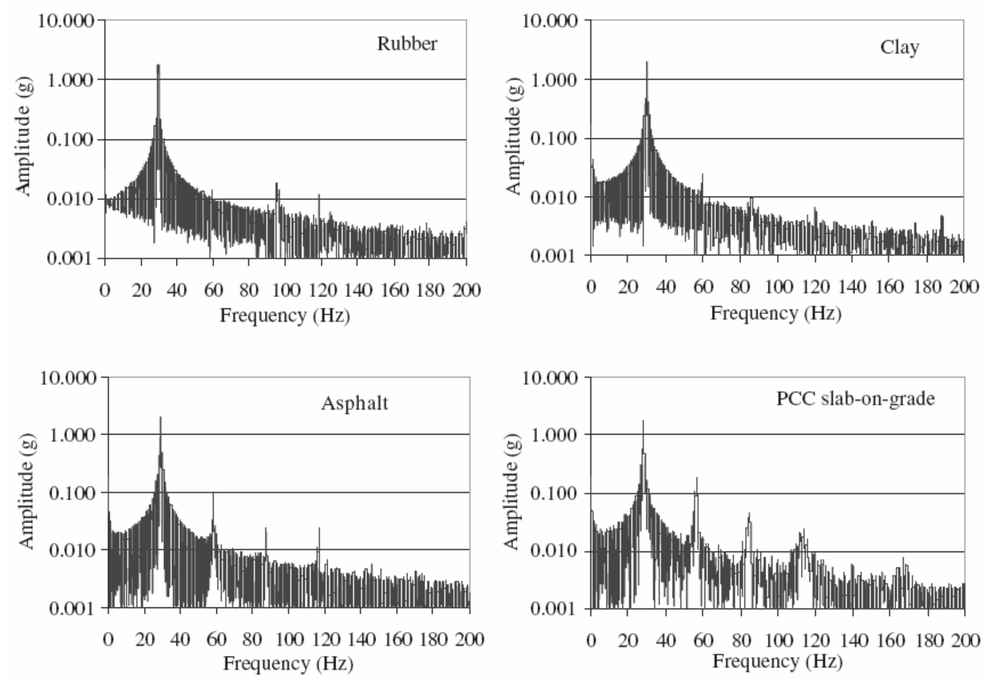


Figure 4.20 – Fourier Spectra of different materials. High THD can be associated with a stiff ground (Mooney et al., 2005).

Figure 4.21 presents a correlation of THD with DCP test on granular materials where the THD is sensitive to the soil type.

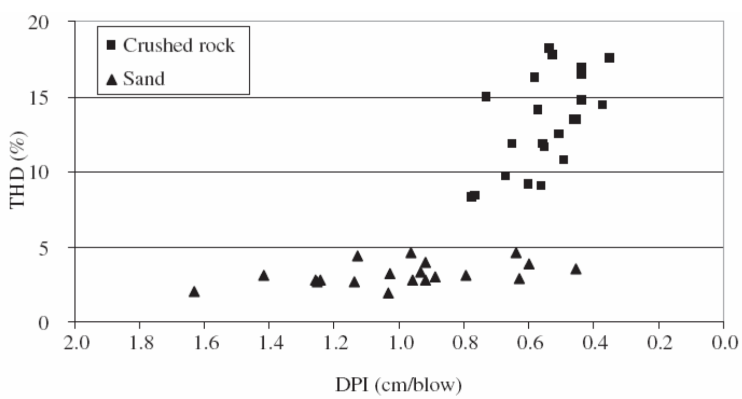


Figure 4.21 – Total Harmonic Distortion during compaction of sands and crush rock (Mooney et al., 2005).

From their field test analysis, Mooney et al., (2005) point out the following important aspects:

- Soil profile and bedrock position effects show that the peak acceleration and THD are insufficient information to assess the compaction state of soils.

- Fluctuation of vibration frequency, roller travel velocity, local variability of water content, and soil type, and soil profile strongly affect the roller-ground response.

4.5.5 Field large scale test

In support of AASHTO, NCHTP, and FHWA in the implementation of a rational pavement design methods, Departments of Transportation and equipment manufacturers have been working for the last 2 years to evaluate ACT through conducting field tests in the USA. The main findings from two field tests performed in Minnesota and in Florida are discussed.

Minnesota demonstration

The demonstration was carried out at the outdoor pavement laboratory of the Minnesota Department of Transportation. The tested section was 200 ft (60 m) in length and 25 ft (7.6 m) wide, which was excavated about 7 to 8 ft (2 to 2.5 m) deep to remove culvert pipes. The field work is briefly described and the main findings and conclusions are presented.

- Materials: Four soils were used to construct the road foundation. The silty sand (SM) that came from the excavation was used as the lower fill material, over which a poorly graded sand (SP) was placed. Then a railroad ballast material was placed over the previous materials. The railroad ballast was used in an attempt to stiffen the fill. More loamy sand (SP) was brought from offsite and placed over the ballast. Finally, crushed granite was used as aggregated base (See Fig 4.22).

- Field measurements: The compaction was done using the BOMAG roller with a target modulus value of 6,500 lb/in^2 (45 MPa) for the first 6 lifts and 14,500 lb/in^2 (100 MPa) for the remaining four lifts. After the appropriate number of passes in each lane, the roller was moved to adjacent lanes and compaction measures were done with portable devices and equipment that included: Dynamic Cone Penetrometer, GeoGauge, Loadman (portable Light Weight Deflectometer), Nuclear Density Gauge and Plate Load Test. These companion tests were performed at all locations, except the Plate Load Test, which was performed only in two locations.

Figure 4.22 – Construction sequence (Petersen, 2004).

• Main results and observations:

- The 45MPa target modulus for the first lifts, that are the base of the fill, was not achieved because of the high water content of the material due to heavy rainfalls on the previous day.

- No correlation were observed between the BOMAG modulus and the other test results due to stress level, loaded area and shape, dependency of soil modulus on particle size, and heterogeneity of the soils. The BOMAG senses the properties of a large volume of soil, while the companion tests are clustered near the surface and in a small area in the middle of the drum.

  The findings are important in practice because the roller would estimate lower values of modulus than the conventional surface test when the foundation of the road is a soft soil. Conversely, if the foundation soils are stiffer the roller would estimate higher modulus.

- Lack of agreement of vibratory compaction results and the companion field test results were due to the stress dependency of the modulus, difference in boundary conditions, and difference in drainage conditions.

- An aspect that clearly supports the use of IC technology was that despite the high moisture contents in the silty sand (SM) soils, the relative compaction results met the compaction requirement of 95% standard Proctor. However, the modulus decreased for moisture increments beyond the optimum water content (See Section 2.4).

- The compaction record from the IC roller identified the areas that were under compacted. These areas would not be identified through the spot test results due to their limited coverage of the work area.

Florida demonstration

In Florida most of the lime-rock base courses have a maximum particle size of 3 inches and minimum percentage of fines of 35%. Currently, the Florida Department of Transportation (FDOT) allows the compaction lifts up to 6 inches (150 mm). In order to reduce costs, FDOT in collaboration with University of Florida conducted field tests to evaluate the compaction of thicker lifts. The compaction field work was conducted in three adjacent 100 ft-long (30 m-long) sections. Two regular compaction rollers and one IC roller were used in this research.

- Materials: a well graded limerock with limited fines at moisture contents dry of optimum was selected for the test.

- Field measurements: the field work was divided in three sections. The following table presents the equipment used in each work section.

Table 4.1 Equipment used in each work section for Florida demonstration

Sec-tion	Equipment	Description	Weight	Lift thickness
1	Bomag 211-D3	Smooth vibratory roller	53,000 lbf (235 kN)	6 in (150 mm)
2	Bomag 213-PD	Pad foot roller	62,000 lbf (276 kN)	12 in (300 mm)
3	Bomag BW 225BV-3	IC smooth vibratory roller	85,000 lbf (378 kN)	12 in (300mm)

The evaluation of the compaction was done through field tests. Density and water content was estimated with nuclear gauge devices; strength was estimated with dynamic cone penetration test; and stiffness was measured with the FWD and the GeoGauge. Additionally, the three sections were instrumented to measure vertical stresses, accelerations, and strains at different depths within the soil being compacted.

- Main results and observations: The results of field test and monitoring instruments indicated an unsuccessful performance of Intelligent Compaction equipment. Unfortunately, since the main objective of the research was not to evaluate IC technology, no efforts were devoted to study the causes of the problems. The findings related to IC were:

- Section 3, which was compacted with the highest compaction energy using Intelligent Compaction equipment, presented the lowest FWD mean stiffness as well as the highest coefficient of variation.

- The modulus obtained from the IC equipment (E_{vib}) was estimated from the stiffness of the soil considering the drum as a loading device (See Section 4.5.4). The E_{vib} data trend indicated softening of the soil with increasing roller passes. However, based on the field test and monitoring instruments, the opposite was observed.

Demonstrations Conducted by Iowa State

White et al. (2006) reported on three demonstration projects, two at a Catepillar site in Illinois and one in Minnesota, all using Caterpillar equipment developed for CCC using engine energy measurements. The "Primary research tasks involved (1) performing experimental testing and statistical analyses to evaluate machine power in terms of soil compaction and the properties of compacted soil (e.g., density, strength, stiffness), and (2) developing recommendations for using the compaction monitoring technology in practice." Test strips were utilized and correlations were established with a variety of conventional field tests including: soil density (nuclear moisture-density gauge), strength (Dynamic Cone Penetrometer, Clegg impact hammer), and stiffness (Geo Gauge, Portable Falling Weight Deflectometer, Plate Load Test). They recommended that a test strip be used to calibrate the compaction equipment through correlations with conventional test methods. They noted that observed variability from this method was due to variation in the soil properties and to errors associated with measurement error. The stressed the importance of water content in the measurement correlations.

They suggested that implementation of the Compaction Monitoring Technology used in their tests into QC/QA specifications could be done with three different types of specifications: Method, End Result, and Performance-Based. Details are presented by White et al. (2006).

4.6 References

Ageikin, IA.S., 1987. Off-the-road mobility of automobiles, Amerind Publishing Co.Pvt. Ltd., New Delhi. Translated from Russian.

Anderegg, R. and Kaufmann, K. 2004. Intelligent compaction with vibratory rollers. TRB annual meeting.

Bekker, M.G., 1956. Theory of land locomotion, the mechanics of vehicle mobility, Ann Arbor, The University of Michigan Press.

Bekker, M.G., 1969. Introduction to Terrain-Vehicle Systems, Ann Arbor, The University of Michigan Press.

Bell, J.R., 1977 Compaction Energy Relationship of Cohesive Soils - Transportation Research Record, No. 641, p29-34

Briaud, J.L and Seo, J., 2003. Intelligent compaction: Overview and research needs, Research Report, Texas A&M University, College Station Texas

Forssblad, L. 1980. "Compaction meter on vibrating rollers for improved compaction control." *Proceedings of International Conference on Compaction*, Vol. II, Paris, p. 541-546.

Heukelom, W., 1961. "Analysis of dynamic deflexions of soils and pavements," *Geotechnique*, Vol 11, pp. 224-42.

INDOT Standard Specifications, 2006. www.in.gov/dot/div/contracts/standards

Jönsson, A., 2001. Modeling, simulation and experimental investigation of a rammer compactor machine, Luleå University of Technolgy, Sweden.

Lambe, T.W., 1958a. The structure of compacted clay. Journal of the Soil Mechanics and Foundation Division, Proceedings of the American Society of Civil Engineers, SM2. Paper No. 1654.

Lambe, T.W., 1958b. The engineering behavior of compacted clay. Journal of the Soil Mechanics and Foundation Division, Proceedings of the American Society of Civil Engineers, SM2. Paper No. 1654.

McVay, M.C. and Jeongsoon, K., 2005. Evaluating thick lift limerock-base course. FDOT C-7984, Draft final report.

Mooney, M.A., Gorman, P.B. and Gonzalez, J.N. 2005. Vibration-based health monitoring of earth structures. June-41, p137-152.

Olson, R.E., 1963. Effective stress theory of soil compaction. Journal of the Soil Mechanics and Foundation Division, Proceedings of the American Society of Civil Engineers, SM2. p27-45.

Petersen, D. L., 2004. Continuous compaction control MnRoad demonstration. Minnesota Department of Transportation. MN/RC-2005-07, Final report.

Sandström, Å.J. and Pettersson, C.B., 2004. Intelligent systems for QA/QC in soil compaction. TRB annual meeting.

Seed, H.B. and Chan, C.K., 1959. Structure and strength characteristics of compacted clays. Journal of the Soil Mechanics and Foundation Division, Proceedings of the American Society of Civil Engineers, SM5. p87-128.

Thurner, H. F and Sandström, Å. J., 2000. Continuous Compaction Control, CCC. European Workshop, Compaction of soils and granular materials, May 19th p237-246.

White, D.J., Jaselskis, E.J., Schaefer, V.R., and Cackler, E.T., 2005. Real-time compaction monitoring in cohesive soils from machine response. Transportation Research Board, 84th Annual meeting, Washington, DC.

White, D.J., Edward, J.J., Schaefer, V.R., Cackler, E.T., Drew Isaac, and Li, L. 2004. Field evaluation of compaction monitoring technology: Phase I. Iowa DOT project TR-495. Final report.

White, D.J., Thompson, M.J., and Jovaag, Kari. 2006. Field Evaluation of Compaction Monitoring Technology: Phase II, Final Report, Iowa Department of Transportation through Center for Transportation Research and Education, CTRE Project 04-171, March, 459p. http://www.ctre.iastate.edu/reports/compaction_2.pdf

Wong, J.Y. 1993. Theory of ground vehicles, Second Edition, John Wiley & Sons, Inc.

(http://www.pooledfund.org/projectdetails.asp?id=954&status=1
(http://www.tfhrc.gov/pubrds/marapr98/soil.htm).
(http://www.mrr.dot.state.mn.us/research/mnpave/meresource.asp,
(http://www.ctre.iastate.edu/Research/detail.cfm?projectID=190237771)

CHAPTER 5 - CONCLUSIONS, SUMMARY AND RECOMENDATIONS

5.1 Summary and Conclusions

The current QUALITY CONTROL OF compaction in the USA relies on the evaluation of water content and dry density from spot tests. Although, water content, type of compaction, and compaction energy defines the structure of soils, shear strength and stiffness are the two basic parameters that directly define the mechanical response of the structures supported by compacted soils (See Chapter 2). Thus, the definition of a QUALITY CONTROL OF compaction method based on water content and soil mechanical parameters would provide an efficient and reliable tool not only for compaction quality assessment but also for design, making possible the consideration of specific conditions of the materials. This concept has already been extended to the pavement design in the USA. Considerable effort is being conducted by FHWA in the implementation of the mechanistic-empirical pavement design approach and a design guide is being developed by an AASHTO Joint Task Force on Pavements in cooperation with the National Cooperative Highway Research Program (NCHRP) and Federal Highway Administration (FHWA).

The evolution from the design method based only on experience to the methods based on a combination of mechanics theory and field observations led to the use of the resilient modulus (M_R), which is one of the fundamental parameters of the mechanistic-empirical approach.

The determination of M_R from laboratory tests is time consuming and does not provide the information needed during construction to determine whether the resilient modulus of the subgrade, subbase, and base meet the design requirements. Due to the complexity of the test procedure only a few tests are conducted, which makes difficult to evaluate the subgrade and pavement materials under conditions expected in the natural environment for a long period (e.g. soaked and un-soaked conditions, freeze-thaw, etc.). Furthermore, the specimens are prepared in the laboratory using different compaction procedures than the ones used in the field (e.g. different compaction energy, water content, boundary conditions, and material granulometry), and the applied stress field associated to the test does not represent the boundary conditions and stress path in the field.

On the other hand, field tests are a reliable alternative for the determination of M_R based not only on the characteristics of the materials but also on the boundary conditions (e.g. bed rock position, relative stiffness of subsoil layers). Several field tests were reviewed in Chapter 3 and special attention was given to the Falling Weight Deflectometer (FWD) considering the efficacy of this test and that it works in the stress range of the pavement working conditions.

However, field test for determination of dry density, water content and modulus are spot tests that can only be conducted in selected locations. Considering the inherent variability of soil properties, especially in highway projects, where not only bedrock position but also the soil type and soil structure can continuously change along the project, compaction QUALITY CONTROL through spot tests is not a reliable method. Dry density and water content tests are associated with a scale relation between sampled volume and compacted volume typically around 1:2,000,000.

The above presented limitations support a change in the compaction QC/QA process to achieve the following results:

- Reduction of uncertainty by increasing the sampling rate.
- Fast quality control procedures.
- Quality control techniques that are coupled to the construction process, the availability of real-time processed data allowing to continuously adjust the compaction work.

Advance Compaction Technologies (ACT) have been under development for the last 30 years in Europe; more recently they have been studied in the USA. In several European countries like Germany, Austria, and Sweden it has been implemented in construction standards. ACT includes Continuous Compaction Control (CCC), a compaction method that allows compaction control of 100% of the worked area using instrumented rollers. The roller-ground system response is used in order to obtain uniform compaction. CCC includes instrumenting non-vibratory rollers to measure the net energy being imparted to the soil as the roller passes. This energy is related to the degree of compaction. More frequently, vibratory rollers instrumented with accelerometers are used. For these, the dynamic roller-ground system response is monitored and related to quality control criteria.

ACT also includes Intelligent Compaction (IC), which complements CCC by optimizing the compaction work with a control system that uses the information collected to continuously adapt the performance of the compaction equipment (i.e. roller speed, frequency and amplitude of roller motion and drum motion mode) to meet the specified construction standards.

In general, today the ACT includes: Global Positions Systems (GPS), machine sensors, microprocessors, reliable transduction systems, robust and inexpensive mass storage, and convenient interfaces for accessing and using acquired information. This technology utilizes systems that are attached to the compaction equipment that monitor the equipment performance and use the results to indicate to the operator whether or not the soil beneath the compactor has been compacted sufficiently to meet specifications. With this technology, compaction specifications are likely to transcend the conventional water content and density specifications now in widespread use to those required for the actual performance of pavements and embankments such as resilient modulus, stiffness, and shear strength.

Chapter 4 reviewed the commonly available techniques for static and vibratory compaction and, identified their particular capabilities, advantages, and disadvantages.

CCC technology for static compaction is being developed by Caterpillar. This technology monitors the energy output of the engine and empirically estimates soil properties based on the premise that changes in equipment response are related to the physical properties of material being compacted. The process is a quantified and improved approach to the qualitative method of "listening to how hard the engine is straining" associate with un-instrumented proof rolling.

Evaluation of the Caterpillar technology by Iowa State University (White, D. et al, 2004) shows that the current state of this technology requires considerable refinements before being ready for implementation in practice. Further work is needed in review and analysis of the estimation of net power from the gross power engine, statistical analysis of the power data, GPS mapping of the ground area being compacted, and criteria for compaction degree zonation.

After refinements are made, it is expected that the technology would be constrained to relative measurement of compaction degree. Estimation of strength and modulus parameters that control the earth-structure performance could only be estimated based on preexisting correlations or on correlations obtained for the particular site. This aspect represents the main disadvantage of this technology. The microstructure of the soil is function of soil type, water content, compaction energy, and in clays, the compaction method. Additionally the correlations depend on a non stable parameter, which is the output energy of the engine. This parameter depends on the age and condition of the equipment and on environmental conditions such as temperature which affects equipment lubricants, and can change drastically at the construction site during one day.

On the other hand, CCC technology for vibratory compaction estimates deformability parameters of the soil being compacted considering the drum as a loading instrument, similar to the loading plate of the FWD test. The main components of the instrumentation are accelerometers attached to the roller.

This technology was developed in Europe starting in the seventies and today several equipment manufacturers independently are still improving the technology. One approach calculates soil stiffness (k_s) based on the mathematical analysis of the roller-ground interaction based on clearly identifying the main forces that govern the equilibrium of the system. A second approach estimates a relative parameter called "cylinder deformation modulus E_c" from the frequency domain, evaluating the fundamental and first harmonic components of the roller-ground acceleration, which makes this parameter frequency dependent. These parameters (i.e. k_s and E_c), then can be related to the resilient modulus, by correlating their measurements with field spot tests.

The main European manufactures of CCC and IC are BOMAG from Germany, AMMANN from Switzerland. The firm Geodynamic from Sweden is leading CCC by manufacturing auxiliary equipment that can be easily installed in regular commercial vibratory rollers.

The FHWA and the Minnesota DOT are currently promoting field demonstrations of IC for vibratory compaction. Based on the evaluations of one field test in Minnesota and one in Florida, considerable work is needed in the interpretation of the ACT measurements and their relationship to conventional spot field tests. This aspect is fundamental to the successful implementation of the technology.

The Literature reviewed in this report shows that ACT for vibratory compaction is a promising technology, but additional research is needed before it can be implemented as a robust QUALITY CONTROL process. Calibration and/or validation may continue to require conventional field loading tests and soil moisture determination. Water content during compaction defines the structure of soils.

5.2 *Recommendations*

FHWA, NCHRP and AASHTO have been working in the implementation of the mechanistic-empirical design method in the USA that allows the direct consideration of soil behavior under cyclic loading conditions. Considering the advantages associated with ACT discussed in this report, and the compatibility of it with the mechanistic-empirical design approach, these three important governmental organizations and several state DOTs have expressed their interest in the implementation of Intelligent Compaction through the support of two research projects. Both projects involve extensive field work with the collaboration of IC roller manufacturers.

The first project is known as "Accelerated Implementation of Intelligent Compaction Technology for Embankment Subgrade Soils, Aggregate Base, and Asphalt Pavement Material", and is a pooled fund study led by the FHWA that is co-sponsored by the following state DOTs: GA, IA, IN, KS, MD, MN, MS, ND, NY, PA, TX, and VA. This research project is just getting under way in August 2006. Appendix A contains information on the FHWA Pooled Fund Solicitation 954 (http://www.pooledfund.org/projectdetails.asp?id=954&status=1)and Study No. TPF-5(128) (http://www.pooledfund.org/projectdetails.asp?id=359&status=4).

The second research project entitled "Intelligent Soil Compaction Systems" is led by NCHRP, and is being conducted by Colorado School of Mines and Iowa State University. Appendix B presents a detailed outline of the objectives and scopes of project posted on the NCHRP website.

The main differences between these two research projects are that the first project includes the study of IC performance on asphalt, and involves the active participation of government agencies looking forward the developing of an experienced and knowledgeable IC expertise base within the pool fund participating state DOTs. Coordination between these two projects is intended in order to efficiently use the available resources.

The main objectives of the FHWA and NCHRP research projects include:

- Investigate the IC experience of European government agencies and contractors in order to define how they have implemented and integrated IC to the practice, and what research have they completed or is ongoing among others (See Appendix B).

- Determine the reliability of intelligent compaction systems through extensive field work supported by field instrumentation (e.g. cell pressures, accelerometers, etc) and current QUALITY CONTROL testing devices such as DCP, FWD, and stiffness gauge. The water content in the field will be monitored as well.

- Evaluate the relationship between the IC output deformability parameter and the current QUALITY CONTROL testing devices.

- Evaluate the importance of moisture and bed rock or stiffer layer level on the accuracy of Intelligent Compaction Systems.

- Identify and prioritize needed improvements for IC equipment and current field QUALITY CONTROL testing equipment.

- Based on the previous information formulate a construction specification that address the requirements for real time documentation, discuss the calibration of the IC system and include QC/QA requirements.

The findings of the FHWA and NCHRP research projects will induce a profound change in the design and construction practice of earth-structures and pavements in the USA. It is recommended that INDOT actively participate in the pooled fund project of FHWA and interact with NCHRP. This will allow INDOT to gain knowledge and experience in IC technology, which will be fundamental in the successful implementation of the technology in Indiana through a consistent construction specification that considers INDOT particular requirements.

This synthesis study is recommending both short and long-term actions to be considered by INDOT with the short-term recommendation addressing the transition period until ACT can be implemented and the long-term recommendation to work with FHWA and collaborating NCHRP studies to gain the most benefit from these programs.

5.2.1 Short-term recommendations

The evaluation of IC technology and the formulation of construction specifications by the FHWA and NCHRP projects can take at least 3 to 5 years. Considering the current limitations of the existing specifications and the available tools of INDOT, it is recommended that the proofrolling (PR) technique be used during this transition period in order to reduce the uncertainty of the compaction quality of earth-structures associated to spot test.

Proofrolling is based on a qualitative description of plastic shear strain associated with the pass of the wheels or a drum over a compacted surface. INDOT provided a general specification:

203.26 Proofrolling When proofrolling is specified, the work shall be performed with a pneumatic tire roller in accordance with 409.03(d)3. Other approved equipment such as a fully legally loaded tri-axle dump truck may be substituted for the pneumatic tire roller. There shall be one or two complete coverages as directed. Roller marks, irregularities, or failures shall be corrected.

After the evaluation of the PR specifications of several states DOTs, including Ohio, Minnesota, Wisconsin, New York, Arizona, Iowa, North Carolina and Colorado, improvements to the current INDOT PR specifications should consider:

- When to conduct PR: Conduct the Proof Rolling immediately after compaction in order to assess the material at the compaction moisture.

- How to conduct PR: Specify a range of velocity of the PR equipment, e.g., 2.5 to 10 mph.

- Equipment characteristics: Specify equipment characteristics for Proof Rolling. PR is a qualitative method and standard PR equipment is required for a consistent assessment. The PR equipment requirements of some state DOTs are listed in the table below.

Table 5.1 Equipment characteristics for proofrolling used by States

State	Equipment characteristics
Iowa	Truck loaded to the maximum single legal axel gross weight of 20000 pounds, or the maximum tandem axle gross weight of 34000 pounds.
North Carolina	Evenly spaced 4 rubber tired wheels mounted on a rigid steel frame. The tires should be operated at inflation pressures between 68 and 72 pounds per square inch. A suitable amount of ballast has to be placed on the frame to get a total weight between 48 and 50 tons.
Colorado	Pneumatic tire equipment with a maximum axle load of 18 kips per axle.
New York	Rigid steel frame with a box body suitable for ballast loading up to 45 metric tons gross weight and mounted on four pneumatic tire wheels acting in a single line across the width of the roller on its transverse load center line.

Ohio	Four heavy pneumatic tires wheels, evenly spaced in one line across the width of the roller and mounted on a rigid steel frame capable of support a gross load from 25 to 50 tons. The tires, filled with liquid from 90 to 95% by volume, have to be capable of operating at inflation pressures ranging from 90 to 150 psi. A guide is provided for the adjustment of the load and tire inflation pressure according with the type of material being compacted.

- Failure criteria: In order to minimize personal judgment it is important the definition of a failure criteria. Ohio and Wisconsin considers a failure if the deflection of the surface being compacted is greater than 1 in and 1.5 in, respectively.

Detailed research work in the implementation of a detailed PR specification may not be required considering that PR is a temporary tool, while an IC specification is generated.

5.2.2 Long-term recommendations

Chapter 4 indicated that IC technology is still in evaluation in the USA and additional work is still needed. The active participation of INDOT in the pool funded project of FHWA is highly recommended. The involvement of contractors of Indiana also is recommended to demonstrate the benefits associated with the technology to them.

Based on the detailed review of the mechanical response of pavements under working conditions, the capabilities of the current field tests, and the ACT technology presented in this report, it is recommended that INDOT, in its active collaboration with the FHWA project, consider:

- Definition of strain and stress level associated with the roller for typical subgrade and base materials. This aspect is important in the interpretation of IC results, and their correlation with the results of other loading field tests. Considering that the stress ranges associated to the IC and the current field loading tests differ by more than an order of magnitude, it should be expected soil modulus (stiffness) determined by the methods would differ.

- Evaluation of the water content in the correlation between the IC technology and current field loading test, considering that these two tests are conducted are different times.

- Evaluation of the effect of bed rock position on the estimation of stiffness from the roller. The term bed rock can also apply to an extremely stiff layer respect to the surrounding material that may cause reflection of stress waves. Considering the difference not only in size but also in energy between the roller and the FWD or other field loading tests, suggests that high scatter will occur in correlations between these two tests. A sound understanding of the parameters that determined the response of these systems is required in the proper interpretation of results.

- Analytical studies of FWD test results has revealed the importance of frequency on the ground response (the ground can amplify the response when frequencies are near the fundamental frequencies of the soil). This observation is important in the correlation of IC and FWD results.

- Determine the depth to bedrock or stiff layers. It will be helpful to define of a procedure for estimating rock position from the IC instrumented roller and FWD test. Chang et al. (1992) presented a simple method for the estimation of subgrade thickness (bedrock position) based on the analysis of the free vibration response of the pavement structure. It will be important to evaluate the capability of this simple method.

- The range in which dynamic effects are important for the roller and other dynamic field test (e.g., FWD and GeoGauge) should differ because of the difference in the type of load (harmonic sustained and transient), magnitude of load, difference in the excitation frequency (Roller excitation frequency is normally around 25 Hz to 30 Hz. The FWD pulse has a primary frequency around 15 to 20 Hz, and the GeoGauge excitation varies between 100 and 200 Hz), and the depth of influence of the test method. These aspects are very important in establishing correlations between Advance Compaction Technology and field tests.

- Based on the review of the preliminary outline of the FHWA project, it is observed that the ACT evaluation will be focused on IC technology. It is recommended to include CCC technology developed by Geodymanik, considering that this technology can be adapted to regular rollers. This may considerable facilitate the gradual implementation of IC in the USA.

It is suggested that the FWD test directly on the compacted soil be used by INDOT for the calibration of IC technology. Some items to be considered in doing so include:

- Due to the marked difference between the stiffness of the subgrade and the FWD loading plate, the estimated modulus from FWD is less precise due to the non uniform stress distribution under the loading plate.

- The stiffness of the plate load rubber controls the stress distribution. The softer the rubber the more uniform the stress distribution at the loading plate-surface interface contact. This aspect supports the requirement of research in the application of FWD on soils, where it may be necessary to use a much softer rubber than the one currently being used or an articulated loading plate.

- In order to avoid the effect of plastic deformation of the subgrade and/or base, four conditioning load drops should be applied.

- Due primarily to the confining stress applied by, and stress attenuating effects of the overlying pavement, the subgrade modulus derived from a FWD test that is performed directly on the subgrade or on the base will be smaller than the one obtained for FWD test performed on the pavement after paving.

ACKNOWLEDGEMENTS

The study on Advanced Compaction Quality Control (Project: SPR 22928) was supported by the Joint Transportation Research Program administered by the Indiana Department of Transportation and Purdue University. The contents of this paper reflect the views of the authors, who are responsible for the facts and the accuracy of the data presented herein, and do not necessarily reflect the official views or policies of the Federal Highway Administration and the Indiana Department of Transportation, nor do the contents constitute a standard, specification, or regulation. The authors are grateful to the Federal Highway Administration/ Indiana Department of Transportation/ Joint Transportation Research Project for supporting this research. The authors acknowledge the assistance provided in this research by D. H. Kim (Project Administrator), and Study Advisory Committee Members: Som Hiremath (INDOT), John Kaiser (INDOT), and Lee Gallivan (FHWA).

Source: http://www.pooledfund.org/projectdetails.asp?id=954&status=1

Solicitation Number:	954
Status:	End Solicitation Phase
Title:	Accelerated Implementation of Intelligent Compaction Technology for Embankment Subgrade Soils, Aggregate Base and Asphalt Pavement Material
Sponsoring Agency:	Federal Highway Administration
Sponsor Solicitation Contact:	Fred Faridazar (fred.faridazar@fhwa.dot.gov) Phone: 202-493-3076
Lead Agency Contact:	Tom Harman (tom.harman@fhwa.dot.gov) Phone: 410-962-0134 Fax: 410-962-3655
Lead Agency:	Federal Highway Administration
Study Number:	TPF-5(128)
FHWA Technical Liaison:	Tom Harman (tom.harman@fhwa.dot.gov) FHWA Routing Symbol: HRC-BAL
Partners:	GA, IA, IN, KS, MD, MN, MS, ND, NY, PA, TX, VA
Date Posted:	6/15/2005 1:54:16 PM
Solicitation Expires:	6/15/2006 1:54:16 PM
Commitment Start Year:	2005
Commitment End Year:	2008
Duration:	36 months
100% SP&R Approval:	Approved
Commitments Required:	$350,000
Commitments Received:	$725,000
Background:	The compaction process is a vital final step in the construction of quality, long lasting subgrade soils and pavement materials. Embankments, Subgrades, Base Materials, and Pavement must be well compacted to obtain uniform, optimum density levels that ensure adequate support and strength. Currently used compaction equipment and processes can too often result in inadequate and / or non-uniform material density, which can contribute in short embankment and/or pavement service life. Compaction rollers with intelligent compaction (IC) capabilities have been developed and are routinely used in parts of Europe and Asia. Many studies have shown that the use of IC

technology can dramatically improve the compaction process. Specifically, it seems that the implementation of IC technology may result in more uniform material density, improve the efficiency of compaction operations by reducing the number of passes needed to obtain specification density and can provide a valuable tool in QC/QA by allowing a visual record of material stiffness values at 100% of the roadway locations recorded during compaction. Rollers with IC technology for soils / aggregate (single drum) and asphalt pavement (tandem drum) compaction are now becoming available in the United States. At the same time, FHWA and state DOTs have expressed interest in conducting studies to accelerate the study and implementation of IC technology. To this end, FHWA has produced a report titled "Strategic Plan for Intelligent Compaction" that establishes a five-year plan to study IC, write AASHTO-style construction QC specifications and implement the technology. The report suggests, among other things, that a coordinated effort by roller manufacturers and government agencies be undertaken to use IC technology on various roadway construction projects at locations around the country. An IC Strategic Forum was held in December that included FHWA, equipment manufacturers and state DOT representatives. At that meeting, it was found that some major roller manufacturers were planning to provide a limited number of rollers to the US market and that a number of state DOTS were planning projects to utilize and study IC technology. Based on those two facts, at least five state DOTs in attendance expressed interest in participating in a pooled fund approach to coordinate the study and rapid implementation of IC technology.

For more background information see the brochure, Intelligent Compaction: Overview and Research Needs, at http://www.webs1.uidaho.edu/bayomy/trb/afh60/IntCompaction_Briaud_September2004_.pdf

Objectives: The primary outcome of the pooled fund project will be:

1. Accelerated development of Intelligent Compaction (IC) QC/QA specifications or Subgrade Soils, Aggregate Base and Asphalt Pavement Material. The focus of the specifications will be to provide a reliable method to capture the maximum potential value added which is possible from current IC technology, and current used/available QC/QA Field testing equipment (dynamic cone, FWD, Plate Load Tests, Density, Moisture, temperature, etc.). Not all possible the potential IC value. All that is possible using current IC and QC/QA Field testing technology.

2. Develop an experienced and knowledgeable IC expertise base within Pool Fund Participating State DOTs.

3. Identify and prioritize needed improvements to, and/or

research for, IC equipment and Field QC/QA testing equipment. Prioritization will be based on the potential for: (1) simplifying IC usage; (2) achieving greater IC value, cost benefit, etc.; (3) higher accuracy; and (4) any combination of 1 through 3.

Scope of Work: 1. Develop Report "Intelligent Compaction in Europe: The Owners Experience and Perspective." Currently, all data, exposure, knowledge, and perspectives have been provided by the IC Equipment Manufacturers. Implementation of IC within the US could be greatly accelerated by a documented report on the European owners IC experiences, perspective, and active research activities. Several of the questions to be answered by the report include:

· Why has/does the owner use IC?
· What qualitative or quantitative value do they get?
· How have they successfully implemented and integrated IC?
· What research have they completed or is ongoing? Can we collaborate so that we can leverage our resources? They do one part we do another, and we share.
· Is there a way to establish a broader based users group for moving technology and testing forward?
· Can we collect information on QA/QC, testing equipment, methods etc?

Advanced compaction technology and methods have been used in Europe by highways, airports, and high-speed rail. Leaders in this area have been the Swedes and Germans who began advanced compaction techniques and in the mid 70's and has had specifications in place for over a decade. In addition, the French have advanced compaction testing equipment that may well be superior to the Germans or Swedes. The Report Team will focus on embankments, subgrades, and non-bound base materials.

2. Conduct integrated multi-state IC construction projects (not limited scope equipment demonstrations) to answer key questions about the technology. The goal is for each Pool Fund Participating State to gain experience and expertise from each IC project regardless of its location within the US. Engineers from Pool Fund Participating State will work as a virtual team on each new IC project. Building and sharing IC knowledge with each new project. The goal is for each DOT to gain significantly more IC knowledge via this method then they would have obtained if an equivalent number of IC projects where performed in their home state. In addition to cost savings, this approach should radically reduce the time required to develop IC specifications and development of a US based IC expert pool and network.

3. Providing a travel mechanism for Pool Fund IC engineers to participate in IC business meetings and IC construction projects in fellow participating States.

4. Plan of Action will include a Pool Fund facilitator to assist DOTs with project planning, scheduling and data collection and to coordinate with roller suppliers to schedule the right equipment at the right location at the right time. It is

envisioned that the facilitator will be paid consultant.

Comments: Each state participating in the study is asked to contribute a minimum of $25,000 per fiscal year.

Documents: http://www.pooledfund.org/documents/solicitations/954.pdf

Subjects: Materials and Construction

Source: http://www.pooledfund.org/projectdetails.asp?id=359&status=4

Study Number: TPF-5(128)

Status: Cleared by FHWA

Title: Accelerated Implementation of Intelligent Compaction Technology for Embankment Subgrade Soils, Aggregate Base and Asphalt Pavement Material

Contract/Other Number:Sponsoring Agency: Federal Highway Administration

Lead Agency: Federal Highway Administration

Lead Agency Contact: John D'Angelo (john.d'angelo@fhwa.dot.gov)
Phone: 202-366-0121

FHWA Technical Liaison: Tom Harman (tom.harman@fhwa.dot.gov)
FHWA Routing Symbol: HRC-BAL

Study Partners: GA, IA, IN, KS, MD, MN, MS, ND, NY, PA, TX, VA

Contract Amount: $725,000

Commitments Received: $725,000 **100%**

SP&R Approval: Approved

Background: The compaction process is a vital final step in the construction of quality, long lasting subgrade soils and pavement materials. Embankments, Subgrades, Base Materials, and Pavement must be well compacted to obtain uniform, optimum density levels that ensure adequate support and strength. Currently used compaction equipment and processes can too often result in inadequate and / or non-uniform material density, which can contribute in short embankment and/or pavement service life. Compaction rollers with intelligent compaction (IC) capabilities have been developed and are routinely used in parts of Europe and Asia. Many studies have shown that the use of IC technology can dramatically improve the compaction process. Specifically, it seems that the implementation of IC technology may result in more uniform material density, improve the efficiency of compaction operations by reducing the number of passes needed to obtain specification density and can provide a valuable tool in QC/QA by allowing a visual record of material stiffness values at 100% of the roadway locations recorded during compaction. Rollers with IC technology for soils / aggregate (single drum) and asphalt pavement (tandem drum) compaction are now becoming available in the United States. At the same time, FHWA and state DOTs have expressed interest in conducting studies to accelerate the study and implementation of IC technology. To this end, FHWA has produced a report titled "Strategic Plan for Intelligent Compaction" that establishes a five-year plan to study IC, write AASHTO-style construction QC specifications and implement the technology. The report suggests, among other things, that a coordinated effort by roller manufacturers and government agencies be undertaken to use IC technology on various roadway construction projects at locations around the country. An IC Strategic Forum was held in December that included FHWA, equipment manufacturers and state DOT representatives. At that meeting, it was found that some major roller manufacturers were planning to provide a limited number of rollers to the US market and that a number of state DOTS were planning projects to utilize and study IC technology. Based on those two facts, at least five state DOTs in attendance expressed interest in participating in a pooled fund approach to coordinate the study and rapid implementation of IC technology.

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Comments: Each state participating in the study is asked to contribute a minimum of $25,000 per fiscal year.

Subjects: Materials and Construction

National Cooperative Highway Research Program
Project 21-09, FY 2006
Intelligent Soil Compaction Systems

Res. Agency:	Colorado School of Mines
Principal Invest:	Michael Mooney
Effective Date:	August 30, 2006
Completion Date:	February 28, 2009
Funds:	$599,879
NCHRP Staff:	Walter Diewald

BACKGROUND

Compaction of embankment, subgrade, and base materials is a significant portion of state highway construction budgets and is critical to the performance of highway pavements. Heterogeneity of earth materials, variability in equipment and operators, and difficulty in maintaining uniform lift thickness and prescribed moisture content combine to make desired earthwork compaction difficult to achieve. Current quality-control and quality-assurance testing devices--such as the nuclear gage, the dynamic cone penetrometer, the stiffness gauge, and the lightweight falling weight deflectometer--are typically used to assess less than one percent of the actual compacted area. In addition each of these testing devices measures values unique to the device.

Intelligent soil compaction has the potential to improve infrastructure performance, reduce costs, reduce construction duration, and improve safety. Intelligent soil compaction involves: (a) continuous assessment of mechanistic soil properties (e.g., stiffness, modulus) through compaction-roller vibration monitoring; (b) continuous modification of roller vibration amplitude and frequency, and (c) an integrated global positioning system to provide a complete GIS-based record of the earthwork site.

Research findings in Europe and in the United States have shown that soil stiffness and modulus can be assessed through vibration of the compaction roller drum and that continuous monitoring, feedback, and automatic adjustment of the compaction equipment can significantly improve the quality of the compaction process. Standard specifications for the application of intelligent compaction systems in the United States are needed. Such specifications should build on existing specifications and experience gained in Germany, Switzerland, Finland, Sweden, Japan, and other countries.

OBJECTIVES

The objectives of this research are to determine the reliability of intelligent compaction systems and to develop recommended construction specifications for

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the application of intelligent compaction systems in soils and aggregate base materials. (See Special Note B.)

Accomplishment of the project objective will require at least the following tasks.

TASKS

Task descriptions are intended to provide a framework for conducting the research. The NCHRP is seeking the insights of proposers on how best to achieve the research objectives. Proposers are expected to describe research plans that can realistically be accomplished within the constraints of available funds and contract time. Proposals must present the proposers' current thinking in sufficient detail to demonstrate their understanding of the issues and the soundness of their approach to meeting the research objectives.

PHASE I

(*1.*) Conduct a review of domestic and international literature and determine the current state of practice of intelligent compaction of soils and aggregate base materials. Identify and translate foreign language specifications and literature deemed to be applicable and especially useful in achieving the project objective.

(*2.*) Query compaction equipment manufacturers and collect pertinent intelligent compaction roller data to determine equipment capabilities and the current state of practice. The data should include modulus, acceleration, amplitude, frequency, speed, contact area, compactive effort, efficiency, global position, and displacement.

(*3.*) Coordinate with state departments of transportation (DOTs) and construction contractors and identify active construction projects for the purpose of scheduling field operations for the collection and comparison of intelligent and traditional compaction data. Coordinate with at least five state DOTs and give special attention to selecting construction projects that provide a wide variety of soil types.

(*4.*) Visit one of the construction projects identified in Task 3. Formulate a data collection plan and collect roller data, instrumentation data, and in-situ testing data. The roller data should be continuously recorded and include modulus, acceleration, amplitude, frequency, speed, contact area, compactive effort, efficiency, global position, and displacement. Instrumentation data should include measurements from buried in ground sensors such as strain gauges, accelerometers, and bender elements for verifying the reliability of the intelligent compaction roller data. In-situ tests should include plate load, dynamic cone penetrometer, light weight deflectometer, nuclear density gage, stiffness gage,

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falling weight deflectometer, sand-cone, and other appropriate tests. In-situ data should include moisture content.

(*5.*) Analyze the Task 4 data and validate the roller data with the instrumentation data. Correlate the roller data with the in-situ data. Determine the importance of moisture, layer depth, and the foundation layer on the accuracy of intelligent compaction systems.

(*6.*) Based on the Task 5 analysis, provide acceptability criteria for establishing target values for modulus. Develop preliminary specifications for the intelligent compaction of soils and aggregate base materials.

(*7.*) Submit an interim report documenting the work completed in Tasks 1 through 6. The interim report should include the preliminary specifications for review by the NCHRP panel. Include an updated, detailed work plan for completing Phase II of the research as a separate appendix to the interim report. Meet with the NCHRP panel to discuss the interim report and Phase II work plan. Work on Phase II will not begin without approval of the NCHRP panel.

PHASE II

(*8.*) For the construction projects identified in Task 3 and not yet visited, formulate a data collection plan and collect roller, instrumentation, and in-situ testing data. The roller data should be collected from a minimum of three different intelligent compaction roller manufacturers. The roller data should be continuously recorded and include modulus, acceleration, amplitude, frequency, speed, contact area, compactive effort, efficiency, global position, and displacement. Instrumentation data should include measurements from buried in ground sensors such as strain gauges, accelerometers, and bender elements for verifying the reliability of the intelligent compaction roller data. In-situ testing devices should include, plate load tests, dynamic cone penetrometer, light weight deflectometer, nuclear density gage, stiffness gage, falling weight deflectometer, sand-cone test, and other appropriate tests. In-situ data should include moisture content.

(*9.*) Analyze all the collected data and validate the roller data with the instrumentation data. Correlate the roller data with the in-situ data. Determine the importance of moisture, layer depth, and the foundation layer on the accuracy of intelligent compaction systems. Based on the analysis of the acquired data, provide target values for the modulus of different soil types.

(*10.*) Develop recommended construction specifications for the application of intelligent compaction systems in soils and aggregate base materials.

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(*11.*) Submit a final report documenting the entire research effort. The final report should address the reliability and effectiveness of intelligent compaction technology in different soil types. The Task 10 construction specifications should be included as a separate appendix.

SPECIAL NOTES

No more than 10 months should be spent in the completion of Phase I. (The 10 months includes 2 months for review and approval of the interim report and the Phase II work plan).

For the purposes of this project, intelligent compaction is defined as continuous calculation of modulus with a real-time feedback mechanism and automatic adjustment. Intelligent compaction involves the use of rollers that are equipped with a control system that can automatically adjust compactive effort in response to a materials modulus during the compaction process. The roller must also be equipped with a documentation system that allows continuous recordation such as the number of roller passes and roller-generated material modulus. The output must (1) enhance the ability of the roller operator and/or project inspection personnel to make real-time corrections in the compaction process; (2) be available for inspector review on demand; (3) allow for a plan-view, color-coded plot of roller stiffness and/or roller pass number measurements throughout a designated section of roadway.

The state Departments of Transportation for Minnesota, Florida, North Carolina, Colorado, and Maryland have volunteered to assist in the identification of active construction projects for the on-site data collection and in-situ testing efforts. Points of contact for each state are available from NCHRP after contract award.

The research team should include technical expertise to assist in the incorporation of international state of the art technology and translation of foreign language specifications and literature.

The Intelligent Compaction Specification developed under this project should:

1. Address real time documentation. The real time documentation for acceptance should be a simple graphical and text presentation recording the modulus value in relation to a specified value. The presentation should be in 10 cm by one drum width blocks. Position identification should be recorded at both ends of the roller using GPS technology.

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2. Be generic. The specification should be generic with respect to the type of compaction equipment (i.e., vibratory, static, smooth, sheep's foot, and pneumatic rollers).
3. Address minimum equipment size and required compactive effort.
4. Discuss the calibration of intelligent compaction equipment.
5. Include Quality Control and Quality Assurance requirements.

NCHRP Project 10-65, Nondestructive Testing Technology for Quality Control and Acceptance of Flexible Pavement Construction, is currently underway, but scheduled to be completed before the start of this research. The research team should use the test protocols recommended in NCHRP Project 10-65 as much as possible in the data collection effort for this project. An interim report for Project 10-65 is available from NCHRP upon request by emailing Ms. Patricia Heard at pheard@nas.edu.

Proposals should include a task-by-task breakdown of labor hours for each staff member as shown in Figure 5 in the brochure, "Information and Instructions for Preparing Proposals" (http://trb.org/nchrp under "Current RFPs [Requests for Proposals]"). Proposals also should include a breakdown of all costs (e.g., wages, indirect costs, travel, materials, and total) for each task.

NCHRP projects are intended to produce results that will be applied in practice, and proposals and the project final report must contain implementation plans for moving the results of the research into practice. Item 4(c), "Anticipated Research Results," in each proposal must include an Implementation Plan that describes activities to promote application of the product of this research. It is expected that the implementation plan will evolve during the project; however, proposals must describe, as a minimum, the following: (a) the "product" expected from the research, (b) the audience or "market" for this product, (c) a realistic assessment of impediments to successful implementation, (d) the institutions and individuals who might take leadership in applying the research product, (e) the activities necessary for successful implementation, and (f) the criteria for judging the progress and consequences of implementation.

Item 5 in the proposal, "Qualifications of the Research Team," must include a section labeled "Disclosure." Information relevant to the NCHRP's need to ensure objectivity and to be aware of possible sources of significant financial or organizational conflict of interest in conducting the research must be presented in this section of the proposal. For

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example, under certain conditions, ownership of the proposing agency, other organizational relationships, or proprietary rights and interests could be perceived as jeopardizing an objective approach to the research effort, and proposers are asked to disclose any such circumstances and to explain how they will be accounted for in this study. If there are no issues related to objectivity, this should be stated.

Funds Available: $600,000

Contract Time: 30 months (includes 2 months for NCHRP review and approval of the interim report, and 3 months for NCHRP review and for contractor revision of the final report)

Staff Responsibility: Mr. Timothy Hess, 202/334-2049 (E-mail: timhess@nas.edu)

Authorization to Begin Work: June 2006--estimated